CHAPTER 10

BRIDGE HYDRAULICS

Chapter 10 Bridge Hydraulics
Table Of Contents

Chapter 10 Bridge Hydraulics
Table Of Contents

10.1 Introduction

10.1.1 Overview

This chapter provides general design guidance for the design of stream crossing system utilizing bridges through:

- presentation of the appropriate design philosophy, goals, and considerations.

- discussion of the technical aspects of hydraulic design including a design procedure which emphasizes hydraulic analyses using the computer program HEC-RAS.

- presentation of equations and methodology for scour analysis.

Waterway bridges are structures that carry traffic over a waterway: the stream crossing system includes the approach roadway, relief openings, when present, and the bridge structure. A more in-depth discussion of the philosophy of Bridge Hydraulics is presented in the AASHTO Highway Drainage Guidelines, Chapter VII(1). Hydraulics of culverts used for stream crossings should be analyzed in accordance with Chapter 8, Culverts.

10.1.2 Objectives

The objective in performing a hydraulic design of a stream crossing system is to provide a cost-effective crossing that satisfies criteria regarding the desired level of hydraulic performance at an acceptable level of risk. The relevant areas for consideration are impact on the stream environment, hydraulic performance, and total economic costs for the stream crossing system: the total cost includes construction, maintenance and risk costs of traffic delay, repair and liability.

The desired level of hydraulic performance can be generally described as:

- to not adversely impact adjacent properties
- to not significantly adversely impact the stream and its environment outside the project
- to have the bridge withstand all flow events up to and including a “Superflood” event

These are further quantified in section 10.3. To meet the stated goals, a hydraulic analysis of the stream crossing system must be performed. The questions to be addressed are:

1.) Determine the changes in stream behavior at the project-crossing site. This includes both the naturally occurring changes as part of the stream morphology and those related to the construction of the project.

2.) The methods used to quantify the changes include a multi-level geomorphic analysis, a steady-state hydraulic analysis, and prediction of scour at the project site.

The design of a cost-effective stream crossing system requires a comprehensive engineering approach that includes data collection, formulation of alternatives, selection of the most cost effective alternative satisfying the established criteria, and documentation of the final design. Water surface elevations and profiles affect the highway and bridge design and are the mechanism for determining the effect of a bridge opening on upstream and downstream water levels.

10.2 Definitions, Symbols and Abbreviations

10.2.1 Definitions

Armoring is a natural process by which the stream removes only particles that are smaller than the transport size, leaving particles greater than the displaced size to “armor” the bed. Usually this layer is of only one particle thickness. It is **NOT** to be counted upon to resist scour during design storm events.

Bridge Modeling Cross-sections

Approach cross-section (Section 1). Section describing the conditions upstream of the effects of the roadway embankment.

BU, section describing the conditions at the upstream end of the roadway embankment.

BD, Section describing the conditions at the downstream end of the roadway embankment.

Exit cross-section, section describing the conditions of the effects of the roadway embankment

Backwater (h1) is the increase in water surface at section. For bridge modeling it is measured relative to the normal water surface elevation without the effect of the bridge at the approach cross-section (Section 1). It is the result of contraction and re-expansion head losses and head losses due to bridge piers. Backwater can also be the result of a "choking condition" in which critical depth is forced to occur in the contracted opening with a resultant increase in depth and specific energy upstream of the contraction. This is illustrated in Figure 10-2.

Contraction is the reduction in channel width.

Free surface flow is flow at atmospheric pressure. At a bridge opening, it is assumed until flow depth is 1.1 times the hydraulic depth of the opening.

Hydraulic depth is an equivalent depth of flow. It is determined as flow area divided by top width, A/TW.

Normal water surface is the water surface at a section that would occur if there was no impact from the project.

Pressure flow is flow at a pressure higher than atmospheric. This occurs when an opening is totally under water, when the flow comes in contact with the low chord of the bridge. Also called orifice flow.

Weir flow is flow that is under atmospheric pressure and is flowing over an obstruction.

Scour is the removal of material from the bed or bank of a stream.

Submergence is the ratio of the depth of water downstream side divided by the height of the energy grade line above the minimum weir elevation on the upstream side. The submergence must exceed 75% for any reduction in flow to take place.

10.2 Definitions, Symbols and Abbreviations (continued)

10.2.1 Definitions (continued)

Thalweg is the loci of points that describe the bottom of channel flow line.

Steady Flow Water Surface Profiles. The steady flow component in HEC-RAS is capable of modeling subcritical, supercritical, and mixed flow regime water surface profiles. The basic computational procedure is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's equation) and contraction/expansion (coefficient multiplied by the change in velocity head). The momentum equation is utilized in situations where the water surface profile is rapidly varied. These situations include mixed flow regime calculations (hydraulic jumps), hydraulics of bridges, and evaluating profiles at river confluences (stream junctions).

10.2 Definitions, Symbols and Abbreviations (continued)

10.2.2 Symbols

Symbol	Description	Units
A	Area of cross-section of flow	ft^2
d	Depth of flow	ft.
L_{1-2}	Length between section 1 and section 2	ft.
L_{2-3}	Length between section 2 and section 3	ft.
L_{3-4}	Length between section 3 and section 4	ft.
n	Manning's roughness coefficient	-
P	Wetted perimeter	ft.
Q_{mc}	Flow in main channel	cfs
$Q_{lob,rob}$	Flow in overbank, either left or right	cfs
Q_w	Flow approaching scour hole within the scour width under consideration	cfs
R	Hydraulic radius, (A/P)	ft.
S_o	Slope of channel	ft./ft.
TW	Top width	ft.
WSE	Water surface elevation	ft.
Y_a	Depth of flow approaching scour hole, relates to Q_w	ft.
Y_o	Depth of flow	ft.

10.2.3 Abbreviations

FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map
NFIP	National Flood Insurance Program

10.3 Hydraulic Design Goals

10.3.1 General Goals

The process to select the recommended stream crossing system uses the evaluation of alternatives for site-specific criteria including capital costs, traffic service, environmental and property impacts, hazard to human life, and hydraulic performance.

The hydraulic performance evaluation of an alternative is based on the success in meeting the goals for the project.

- **Backwater should not significantly increase flood impact to property.**
- **The existing flow distribution is maintained to the extent practicable.**
- **Increased velocities should not damage either the structure or significantly increase impact to adjacent property.**
- **Scour shall not cause the failure of the bridge.**
- **Life cycle costs for construction, maintenance, and operation are minimized.**

These goals, criteria and possible actions to address these goals are further discussed below.

Goal 1. Backwater should not significantly increase flood impact to property.

The interest of other property owners must be considered in the design of a proposed stream-crossing system. Not all stream crossing systems can be designed to economically pass all possible flows without backwater effects, therefore the effects of flows greater than the roadway operational flow should be evaluated. Overtopping of the roadway may be used to control backwater levels for flows greater than the operational flow. Embankment overtopping incorporated into the design should be located well away from the bridge abutments and superstructure.

In delineated floodplains, whenever practicable, the stream-crossing system shall avoid encroachment on the **floodway** within a flood plain. When this is not feasible, modification of the floodway itself shall be considered. If neither of these alternatives is feasible, FEMA regulations for "floodway encroachment where demonstrably appropriate" shall be met.

Backwater/Increases Over Existing Conditions

The effects of the bridge backwater changes in water surface profiles shall be evaluated at the right-of-way line and shall:

- consider the impact on adjacent properties during the passage of the 1% exceedence probability flood for sites not covered by NFIP.

- conform to FEMA regulations for sites covered by the National Flood Insurance Program (NFIP). See Chapter 2 Legal Aspects for discussion of interaction with FEMA.

10.3 Hydraulic Design Goals (continued)

10.3.1 General Goals (continued)

Waterway Enlargement

There are situations where the proposed roadway and structural designs regarding the vertical positioning of a bridge result in a small vertical clearance between the low chord and the ground. Significant increases in span length provide small increases in effective waterway opening in these cases. Although it may seem possible to increase the effective area by excavating a flood channel through the reach, the desired long-term hydraulic performance does not usually result. The design usually fails to address the issue of a stable channel in regards to erosion, scour or aggradation and deposition. The use of waterway enlargement is discouraged. Its use will require approval from ADOT Drainage Section.

Goal 2. The existing flow distribution is maintained to the extent practicable.

Flow Distribution

The conveyance of the proposed stream-crossing location shall be calculated to determine the flow distribution and to establish the location of bridge opening(s). The proposed facility shall not cause any significant change in the existing flow distribution. Relief openings in the approach roadway embankment should be evaluated when there are flow concentration points that would be blocked by a single bridge opening.

Auxiliary Openings

Stream-crossing systems on wide floodplains may result in the need for auxiliary waterway openings. The purpose of the auxiliary openings on the floodplain is to pass a portion of the flood flow in the floodplain when the stream reaches a certain stage. It does not provide relief for the principal waterway opening in the sense that an emergency spillway at a dam does, but has predictable capacity during flood events.

Basic objectives in choosing the location of auxiliary openings include:

- maintenance of flow distribution and flow patterns,
- accommodation of relatively large flow concentrations on the flood plain,
- avoidance of flood plain flow along the roadway embankment for long distances, and
- crossing of significant tributary channels.

The most complex factor in designing auxiliary openings is determining the division of flow between the two or more structures. If incorrectly proportioned, one or more of the structures may be overtaxed during a flood event. Auxiliary openings should usually be generously sized to guard against that inability to adequately determine the flow distribution. The technological weakness in modeling auxiliary openings is in the use of one-dimensional models to analyze two-dimensional flow.

Goal 3. Increased velocities should not damage either the structure or significantly increase impact to adjacent property.

Velocity

The velocity changes should be minimal. Velocities that result in predicted bed or bank erosion shall be addressed. Erosion protection in the form of bed or bank protection shall be included in the stream crossing system. Whenever possible, clearing of vegetation upstream and downstream of the toe of the embankment

10.3 Hydraulic Design Goals (continued)

10.3.1 General Goals (continued)

slope should be avoided. For the operational flow, wire-tied riprap, railbank, or soil cement bank protection is used to protect the roadway embankment or abutment fill slopes. Bank protection design information is presented in Chapter 11.

Spur dikes should be used, as necessary, to align flows, protect roadway embankments, and mitigate the effects of changes in the stream hydraulic behavior. Spur dikes are recommended to align the approach flow with the bridge opening and to mitigate scour for the operational flow at the abutments. They are usually elliptical shaped with a major to minor axis ratio of 2.5 to 1. Their length can be determined according to HDS-1 (2) or by 7 times the depth of abutment scour to be mitigated. Spur dikes and embankments shall be protected by bank protection for flows up to the operational flow.

Goal 4. Scour shall not cause the failure of the bridge.

Scour

Stream effects causing scour for flows up to and including the 500-year event shall be determined. Scour should not be aggravated due to excessive constriction nor improper alignment. If overtopping of the bridge highway system occurs at less than Q500, the overtopping flow and its impact shall also be evaluated. Debris build-up on foundation elements in the flow shall be included in the determination of design scour. The bridge shall withstand the scour effects for the greatest discharge passing through the bridge up-to and including the “superflood” event.

Goal 5. Costs for construction, maintenance, and operation are minimized.

Operational Frequency

Infringement of storm water into the desired freeboard of the roadway determines the operational level of traffic service provided by the facility. Desired minimum levels of operational frequency for travelway inundation of specific roadways are presented in Chapter 600 of The Highway Design Manual.

Bridge Freeboard Clearance

A minimum clearance of 3 ft for Level 1 bridges or 1 foot for Level 2,3 or 4 bridges shall be provided between the design approach water surface elevation and the low chord of the bridge for the selected design alternative.

10.4 General Considerations

10.4.1 Location Considerations

The primary drainage consideration for facility location in highway planning is the evaluation of the impact of floodplain encroachments of a stream crossing. Hydraulic and environmental considerations of highway river crossings and encroachments are presented in the FHWA Highways in the River Environment, Training and Design Manual (1990). The Manual identifies possible local, upstream and downstream effects of highway encroachments.

The principal hydraulic factors to be considered in locating a stream crossing that involves encroachment within a floodplain are:

- river type (straight or meandering),
- river characteristics (stable or unstable),
- river geometry and alignment,
- hydrology,
- hydraulics,
- floodplain flow, and
- economic and environmental concerns.

A detailed evaluation of these factors is part of the location hydraulics study. When a suitable crossing location has been selected, specific crossing components can then be determined. Exact information on these components is usually not developed until the final stage.
When necessary, these include:

- the geometry and length of the approaches to the crossing,
- probable type and approximate location of the abutments,
- probable number and approximate location of the piers,
- estimated depth to the footing supporting the piers (to protect against local scour),
- the location of the longitudinal encroachment in the floodplain,
- the amount of allowable longitudinal encroachment into the main channel, and
- the required river training works to ensure that river flows approach the crossing or the encroachment in a complementary way.

10.4.2 Morphology

A stream is a dynamic natural system that as a result of the encroachment caused by elements of a stream-crossing system will respond in ways that may be unexpected. Among the many hydraulic factors that affect and need to be considered in the design of a stream crossing system are: flood plain width and roughness, flow distribution and direction, bed slope, stream type (braided, straight, or meandering), stream regime (aggrading, degrading, or equilibrium), and stream controls. The hydraulics of a proposed location may affect environmental considerations such as aquatic life, wetlands, sedimentation, and stream stability. The history of the stream must be considered, including assessment of long-term trends in aggradation or degradation.

The inherent complexities of stream stability, further complicated by highway stream crossings, requires a multilevel procedure. The evaluation and design of a highway stream crossing or encroachment should begin with a qualitative assessment of stream stability. This involves application of geomorphic concepts to identify potential problems and alternative solutions. This analysis should be followed with quantitative

10.4 General Considerations (continued)

10.4.2 Morphology (continued)

analysis using basic hydrologic, hydraulic and sediment transport engineering concepts. Such analyses could include evaluation of flood history, channel hydraulic conditions (up to and including, for example, water surface profile analysis) and basic sediment transport analyses such as evaluation of watershed sediment yield, incipient motion analysis and scour calculations.

This analysis can be considered adequate for many locations if the problems are resolved and the relationships between different factors affecting stability are adequately explained. If not, a more complex quantitative analysis based on detailed mathematical modeling and/or physical hydraulic models should be considered. This multilevel approach is presented in FHWA HEC-20. See Chapter 8 for a more in-depth discussion of morphology.

10.4.3 Water Surface Profile Modeling

The hydraulic analysis of bridges includes the computation of the water surface profile, flow and velocity distribution. The results of the analysis are used to evaluate the adequacy of the alternative in meeting design criteria.

Water surface profiles are computed for a variety of technical uses including:

- drainage crossing analysis,
- flood hazard mitigation investigations,
- flood insurance studies, and
- evaluation of longitudinal encroachments.

Hydraulic computations performed for other agencies shall be carefully evaluated for appropriateness in bridge hydraulic analysis. The evaluation shall consider the completeness and level of detail used in the waterway model.

Water Surface Profile Determination

The water surface profile may be determined by using of mathematical one- or two- dimensional models or by physical models. Where flow is essentially two-dimensional in the horizontal plane, a one-dimension model might not provide an adequate analysis of the cross-stream water surface elevations, flow velocities, or flow distribution. A two-dimensional finite element model, FESWMS is available for the analysis of flows at bridge crossing where unusually complicated hydraulic conditions exist. For exceedingly complex sites that defy accurate or practicable mathematical modeling, physical modeling may be appropriate. The constraints on physical modeling are size, cost and time. **Use of modeling methods other than HEC-RAS shall be only with the prior approval of the ADOT Drainage Section.**

1-Dimensional Modeling

In practice, the water surface profile and velocities in a section of river are often predicted using one-dimensional methods such as the standard step method. The USACOE computer program HEC-RAS is the **recommended** method for performing step backwater water surface profile computations of bridges

10.4 General Considerations (continued)

10.4.3 Water surface profile modeling (continued)

and channels in non-prismatic channels where a one-dimensional model is acceptable. HEC-2 was often used to perform water surface profile modeling. HEC-RAS does not always replicate HEC-2 results. Therefore, prior studies performed using HEC-2 should be re-run with HEC-RAS to establish the current condition model.

2-Dimensional Modeling

Two-dimensional modeling may be necessary for many types of steady and unsteady flow problems including multiple opening bridge crossings, spur dikes, flood plain encroachments, multiple channels, and flow around islands. Two-dimensional models are more complex and require more time to set up and calibrate. They require essentially the same type of field data but at a greater density of information as a one-dimensional model. FESWMS is a finite element model that has been developed to analyze flow at bridge crossings where complicated hydraulic conditions exist.

Physical Modeling

Complex hydrodynamic situations defy accurate or practicable mathematical modeling. Physical models should be considered when:

- hydraulic performance data is needed that cannot be reliably obtained from mathematical modeling,
- risk of failure or excessive over-design is unacceptable, or
- research is needed.

The constraints on physical modeling are size (scale), cost, and time.

10.5 Design Procedures & Modeling Considerations

10.5.1 Outline of Procedure

An outline of a design procedure is presented below; this outline shall be modified as necessary to fit the individual site characteristics.

I. Data Collection

A. Survey.

1. Topography
2. Geology and Soil information
3. Highwater marks
4. History of debris accumulation and scour
5. Review of hydraulic performance of existing structures
6. Maps and aerial photographs for understanding of site and historical trends
7. Rainfall and stream gage records
8. Field reconnaissance

B. Hydrologic and hydraulic studies by other agencies.

1. Federal Flood Insurance Studies
2. Federal Flood Plain Studies by the COE, NRCS, etc.
3. Arizona Department of Water Resources and local flood plain studies
4. Hydraulic performance of existing bridges

C. Hydraulic features which influences site response.

1. Other streams, reservoirs, water intakes
2. Structures upstream or downstream
3. Natural features of stream and flood plain
4. Channel modifications upstream or downstream
5. Flood plain encroachments
6. Sediment types and bed forms

D. Environmental features which impact site.

1. Existing bed or bank instability
2. Flood plain land use and flow distribution
3. Environmentally sensitive areas (fisheries, wetlands, etc.)

E. Site-specific Design Criteria

1. Preliminary risk assessment
2. Application of design criteria

10.5 Design Procedures & Modeling Considerations (continued)

10.5.1 Outline of Procedure (continued)

II. Hydrologic Analysis

A. Watershed morphology

l. Drainage area size (attach map)
2. Watershed characterization (soil type, land use and vegetation)
3. Channel geometry and slope

B. Hydrologic computations

l. Discharge for historical flood that complements the high water marks used for calibration
2. Discharges for specified frequencies

III. Hydraulic Analysis

A. Computer model calibration and verification
B. Hydraulic performance for existing conditions
C. Hydraulic performance of proposed designs
D. Scour analysis of proposed designs

IV. Selection of Final Design

A. Cost effective alternative
B. Measure of compliance with established hydraulic criteria
C. Consideration of environmental and social criteria
D. Design details such as bank protection

V. Documentation

A. Complete project record
B. Complete correspondence and reports

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2 Water Surface Profile Analysis at Bridges

The water surface profile computations are interactive and are solved by application of the step-backwater method using a trial and error process of assuming the energy grade elevation, computing the discharge, and comparing with the specified discharge. It is impracticable to perform the hydraulic analysis for a bridge by manual calculations due to the interactive and complex nature of those computations.

The USACOE computer program HEC-RAS is the **accepted** method for performing step backwater water surface profile computations of bridges and channels in non-prismatic channels where a one-dimensional model is acceptable.

Accuracy of computed water surface profiles

Accuracy in computing water surface profiles with the step backwater profile method is affected by:

- data estimation errors resulting from incomplete or inaccurate data collection and inaccurate data estimation,
- errors in accuracy of energy loss calculations depending on the validity of the energy loss equation employed and the accuracy of the energy loss coefficients selected (Manning's n-value is the coefficient measuring boundary friction),
- inadequate length of stream reach investigated, and
- use of cross-section spacing that is too great. This results in inaccurate integration of the energy loss-distance relationship.

10.5.2.1 Classification of Flow and Computational Methods

The following discussion presents the methods and procedures used in HEC-RAS to perform water surface profiles. The user should review the information presented in the HEC-RAS Hydraulics Reference Manual, (Jan 01). The stream-crossing system is subject to either free-surface flow or pressure flow through one or more bridge openings with possible embankment overtopping. The bridge routines in HEC-RAS have the ability to model low flow (Class A, B, C), low flow and weir flow (with adjustments for submergence on the weir), pressure flow (orifice and sluice gate equations), pressure and weir flow, and highly submerged flows. In free-surface flow (low flow), there is no contact between the water surface and the low-girder (or low chord) elevation of the bridge (open channel flow). In orifice flow, only the upstream girder is submerged, while in submerged orifice flow both the upstream and downstream girders are submerged.

Combination Flow

Sometimes combinations of low flow or pressure flow occur with weir flow. In these cases the program uses an iterative procedure to determine the amount of each type of flow. The program continues to iterate until both the low flow method (or pressure flow) and the weir flow method have the same energy (within a specified tolerance) upstream of the bridge (section 3). The combination of low flow and weir flow can only be computed with the energy and Yarnell low flow method.

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.1 Classification of Flow and Computational Methods (continued)

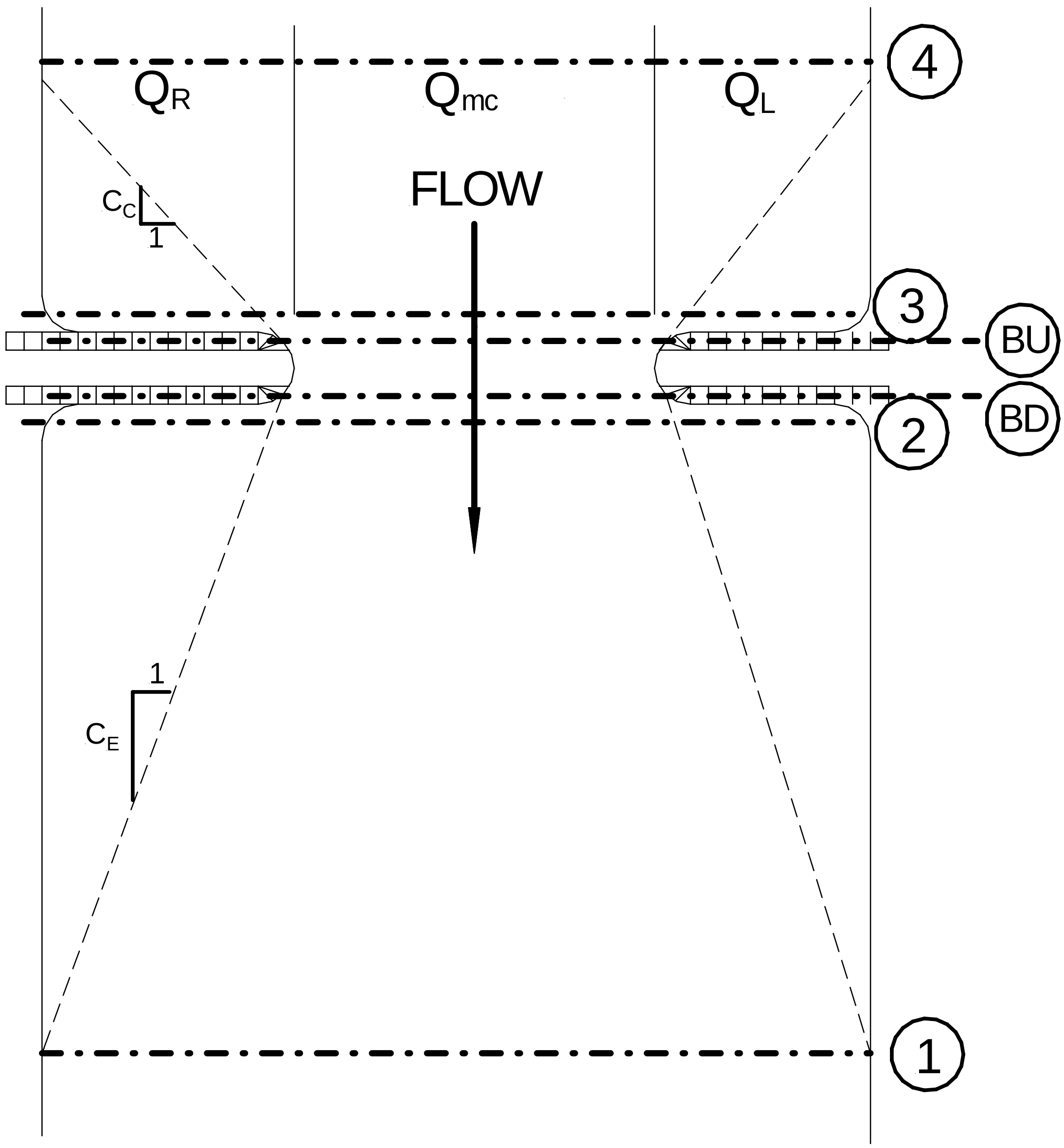


Figure 10.1a - Flow through a Bridge Opening

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.1 Classification of Flow and Computational Methods (continued)

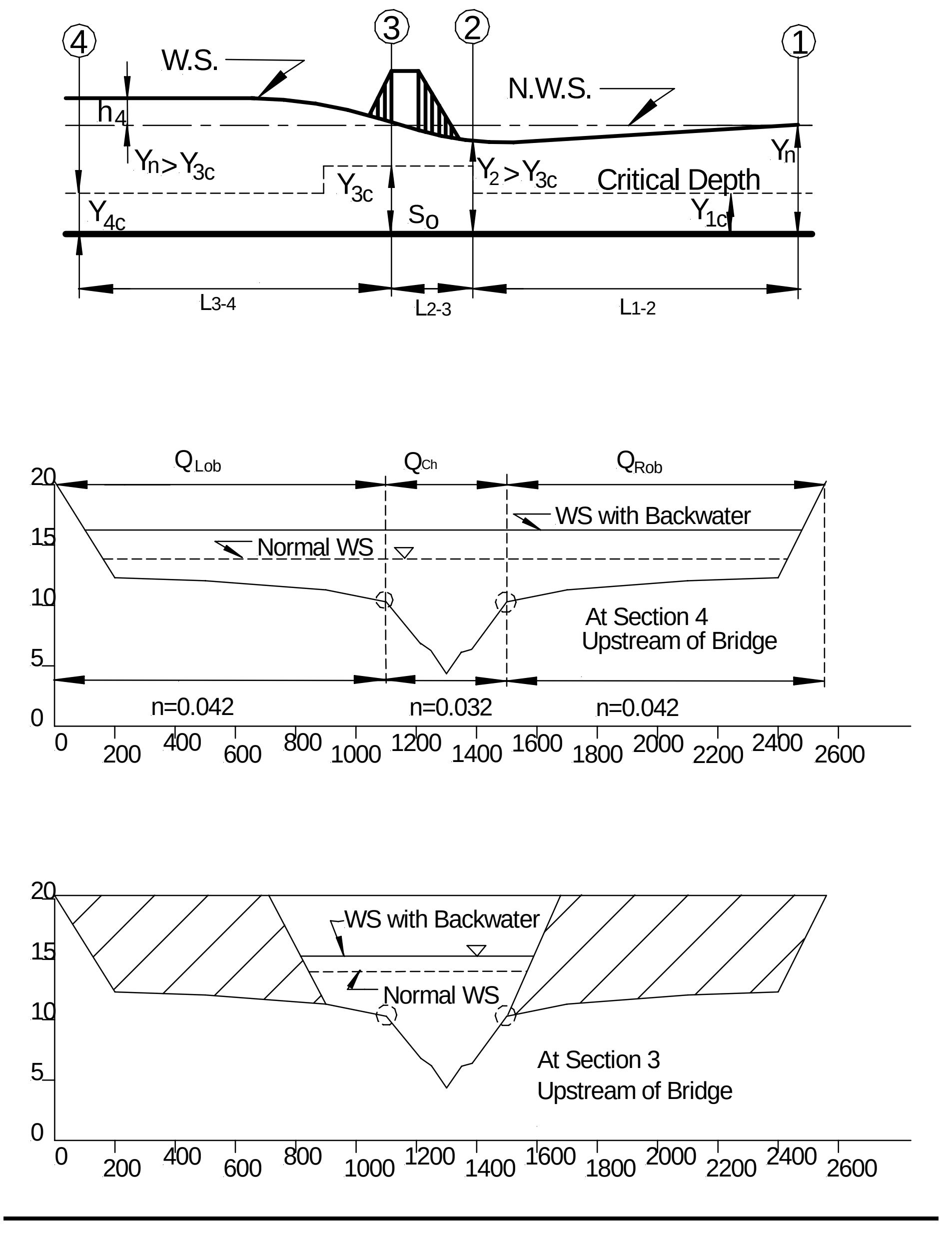


Fig 10.1b Backwater Effect of Bridge

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.1 Classification of Flow and Computational Methods (continued)

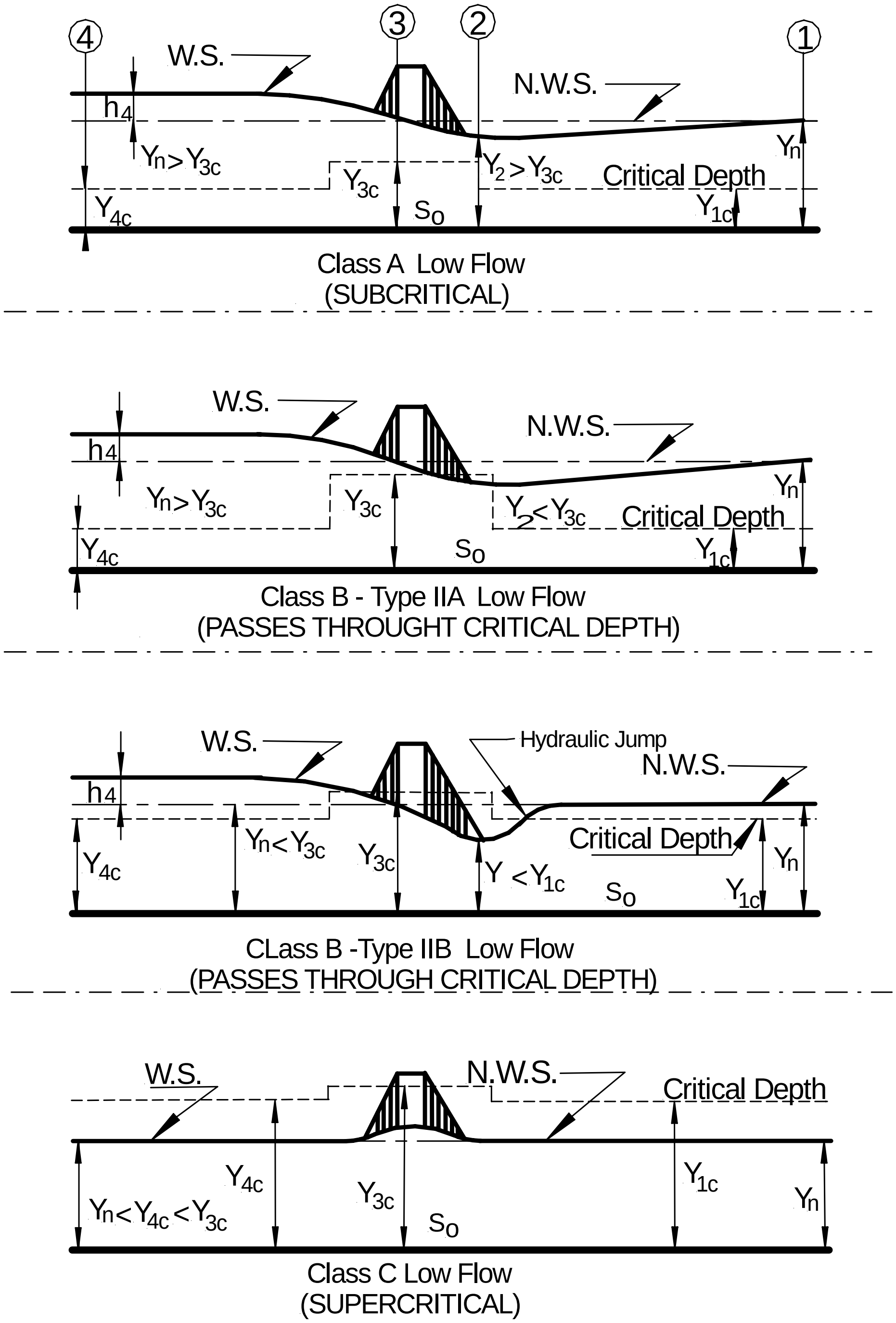


Figure 10-2 Bridge Flow Types

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.1 Classification of Flow and Computational Methods (continued)

10.5.2.1.1. Modeling Cross Sections

The Bridge routines utilize four user-defined cross-sections in the computation of energy losses due to the structure. The cross sections are labeled 1 through 4 in Figure 10-1. During the hydraulic computations the program automatically formulates two additional cross sections inside the bridge (BD and BU).

Cross section 1 is located sufficiently downstream from the structure so that the flow is not affected by the structure. This is usually based on the expansion ratio chosen.

Cross section 2 is located a short distance downstream from the bridge (commonly placed at the downstream toe of the road embankment). This cross section should represent the effective flow area just downstream of the bridge. Cross section 3 is located a short distance upstream from the bridge (commonly placed at the upstream end toe of the road embankment). The distance between cross section 3 and the bridge should only reflect the length required for the abrupt acceleration and contraction of the flow that occurs in the immediate area of the opening. Cross section 3 represents the effective flow area just upstream of the bridge.

Both cross section 2 and 3 will have ineffective flow areas to either side of the bridge opening during low flow and pressure flow profiles. In order to model only the effective flow areas at these two sections, the user should use ineffective flow area options at both these cross sections.

Cross section 4 is an upstream cross section where the flow lines are approximately parallel and the cross section is fully effective.

During the hydraulic computations, the program automatically formulates two additional cross sections inside the bridge structure (BD and BU). The geometry inside the bridge is a combination of the bounding cross sections (sections 2 and 3) and the bridge geometry. The bridge geometry consists of the bridge deck and roadway, sloping abutments if necessary, and any piers that exist.

10.5.2.1.2 Low Flow Computations

Low flow exists when the flow going through the bridge opening is open channel flow (water surface below the highest point on the low chord of the bridge opening). For low flow computations, the program firsts uses the momentum equation to identify the class of flow. The program first calculates the momentum at critical depth inside the bridge at the upstream and downstream ends. The end with the higher momentum (therefore most constricted section) will be the controlling section in the bridge. If the two sections are identical, the program selects the upstream bridge section as the controlling section. The momentum at the controlling section is then compared to the momentum of the flow downstream of the bridge when performing a subcritical profile (upstream of the bridge for a supercritical profile). If the momentum downstream is greater than the critical depth momentum inside the bridge, the class of flow is considered to be completely subcritical. (Class A low flow). If the momentum downstream is less than the momentum at critical depth in the controlling section of the bridge, then it is assumed that the constriction will cause the flow to pass through critical depth and a hydraulic jump will occur at some distance downstream (Class B low flow). If the profile is completely supercritical through the bridge, then it is considered Class C low flow. See Figure 10-2.

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.1.2 Low Flow Computations (continued)

Class A Low Flow:
Class A low flow consists of subcritical flow throughout the approach, bridge, and exit cross sections. It is the most common condition encountered in practice. Energy losses through the expansions are calculated as friction losses and expansion losses. Friction losses are based on a weighted friction slope times a weighted reach length between section 1 and 2. The weighted friction slope is based on one of the four available alternatives in HEC-RAS, with the average conveyance method being the default. The average length used is based on a discharge-weighted reach length. Energy losses through the contraction (sections 3 and 4) are calculated as friction losses and contraction losses; these are calculated in the same way as between sections 1 and 2.

Losses through the bridge are calculated by one of four available user selected methods:

- Energy Equation (standard step method)
- Momentum Balance
- Yarnell Equation
- FHWA WSPRO method

The user can either select the method, or direct the program to use the method that computes the greatest energy loss through the bridge.

Energy Equation
The Energy Equation treats a bridge in the same manner as a natural river cross-section with the area of the bridge below the water surface is subtracted from the total area, and the wetted perimeter is increased where the water is in contact with the bridge structure. The program formulates the two cross sections inside the bridge by combining the ground information of sections 2 and 3 with the bridge geometry. The program begins with standard step calculation at the downstream section 2, then to the downstream section just inside of the bridge, BD, then to the upstream section just inside the bridge, BU, and finally to the upstream section 3. The energy-based method requires Manning's n values for friction losses and contraction and expansion coefficients for transition losses. Detailed output is available for cross sections inside the bridge as well as the user entered cross sections (sections 2 and 3)

Momentum Balance Method
The momentum method is based on performing a momentum balance from cross section 2 to cross section 3. The momentum balance is performed in three steps. The first step is to perform a momentum balance from cross section 2 to the downstream section just inside of the bridge. The equation for this momentum balance is

$$(A*Y+Q^{\wedge 2}/gA)=(A*Y+Q^{\wedge 2}) - A*Y+F-W$$

Area of flow times depth of flow plus Discharge squared divided by gravity times area of flow in the downstream bridge section is set equal to the same quantity in section 2, plus the external force due to friction minus the force due to weight of water in the direction of flow. The second step is a momentum balance between section BD and BU. This has the same form of equation as for the first step. The final step is a momentum balance between section BU and section 3. This step includes a term for the drag for flow going around the piers. The momentum balance requires the use of roughness coefficients of the friction

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.1.2 Low Flow Computations (continued)

Class A Low Flow: (continued)
force and a drag coefficient for the force of drag on the piers. Drag coefficients are used to estimate the force due to the water moving around the piers, the separation of flow, and the resulting wake that occurs downstream. The drag coefficient for square nose piers is 2.0.

The momentum method provides detailed output for the cross sections inside the bridge (BU and BD) as well as outside the bridge (section 2 and 3). The user has the option of turning the friction and weight force components off. The default is to include the friction force but not the weight component. During the momentum calculations, if the water surface at section BD or BU comes in contact with the maximum low chord of the bridge, the momentum balance is assumed to be invalid and the results are not used.

Yarnell Equation

The Yarnell equation is an empirical equation that is used to predict changes in water surface from just downstream of the bridge (Section 2) to just upstream of the bridge (section 3). The drop in water surface from section 3 to section 2 is a function of the ratio of velocity head to depth at section 2, the obstructed area of the piers divide by the unobstructed area at section 2, and the velocity at section 2.

The computed water surface at section 3 is simply the downstream water surface plus the change calculated above. The Yarnell equation is very sensitive to pier shape, pier obstructed area, and velocity of flow. Because of these limitations, the Yarnell method should only be used at bridges where the majority of energy losses are associated with the piers. The pier coefficient for square nose pier is 1.25.

FHWA WSPRO Method

The WSPRO method computes the water surface profile through a bridge by solving the energy equations. The method is an iterative solution performed from the exit cross-section (1) to the approach cross section (4). The energy balance is performed in steps from the exit section (1) to the section just downstream of the bridge (2), from just downstream of the bridge (2) to just inside the downstream end (BD), from just inside downstream (BD) to just inside upstream (BU), from just inside upstream (BU) to just upstream (3), and from just upstream (3) to the approach section (4).

Losses from section 1 to section 2 are based on friction losses and an expansion loss. Friction losses are calculated using the geometric mean friction slope times the flow weighted distance between section 1 and 2. Losses from section 2 to section 3 are based on friction losses only. The energy balance is performed in three steps: from section 2 to section BD, from section BD to section BU, and BU to 3. Friction losses are calculated using geometric mean friction slope time the flow weighted distance between sections.

Losses from section 3 to section 4 are based on friction losses only. This is based on the effective flow length in the approach reach, computed as the average of 20 equal conveyance steam tubes.

Class B Low Flow
Class B low-flow can exist for either subcritical or supercritical profiles. For either profile, Class B flow represents the condition when "choking" of the flow by the bridge opening results in critical flow through the bridge opening. In Type IIA flow, supercritical downstream, the critical water surface elevation in the bridge opening is lower than the undisturbed normal water surface elevation. In the

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.1.2 Low Flow Computations (continued)

Class B Low Flow (continued)

Type IIB flow, subcritical downstream, it is higher than the normal water surface elevation and a weak hydraulic jump immediately downstream of the bridge contraction is possible. For a subcritical profile, the momentum equation is used to compute an upstream water surface (Section 3) above critical depth and a downstream water surface (section 2) below critical depth. For a supercritical profile, the bridge is acting as a control and is as causing the upstream water surface elevation to be above critical depth. Momentum is used to calculate an upstream water surface above critical depth and a downstream water surface below critical depth. If the momentum equation fails to converge on an answer during Class B flow computations, the program will automatically switch to an energy-based method.

Whenever Class B flow is found to exist, the user should run the program in a mixed flow regime mode. If the mixed flow regime is run, the program will proceed with backwater calculations upstream, and later with forewater calculations downstream from the bridge. Any hydraulic jumps that occur either upstream or downstream of the bridge can be located.

Class C Low Flow

Class C flow is supercritical approach flow and remains supercritical through the bridge contraction. The program can use either the energy equation or the momentum equation to compute the water surface through the bridge. Such a flow condition is not subject to backwater unless it chokes and forces the occurrence of a hydraulic jump upstream of the contraction.

10.5.2.1.3 High Flow Computations

The program computes high flow conditions by either the Energy equation (standard step method) or by using separate equations for pressure and/or weir flow.

Energy Equation

The energy-based method is applied to high flows in the same manner as it is applied to low flows. Computations are based on balancing the energy equation in three steps through the bridge. Energy losses are based on friction and contraction and expansion losses. Friction losses are based on use of Manning's equation. Expansion and contraction losses are based on a coefficient times the change in velocity head. The energy-based method performs all computations as they are open channel flow. At the cross sections inside the bridge, the area obstructed by the bridge piers, abutments, and deck is subtracted from the flow area and additional wetted perimeter is added. Occasionally the resulting water surfaces inside the bridge sections (BD and BU) are computed to be at elevations inside the bridge deck. The water surface inside the bridge reflects the hydraulic grade line elevations, not necessarily the actual water surface elevations. The active flow area is limited to the open bridge area.

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.1.3 High Flow Computations (continued)

Pressure Flow Computations

Pressure flow occurs when the flow comes into contact with the low chord of the bridge. Once the flow comes in contact with the upstream side of the bridge, a backwater occurs and orifice flow is established. The program will handle two cases of orifice flow; when only the upstream side of the bridge is in contact with the water, and when the bridge opening is completely full. The program automatically selects the appropriate equations. For the first case, a sluice gate type of equation with a discharge coefficient is used. The program will select the discharge coefficient based on the amount the inlet is submerged.

Free surface flow is assumed to occur until the depth just upstream of the bridge opening exceeds 1.1 times the hydraulic depth of the opening, then pressure flow is assumed. In the second case, when both the upstream and downstream side of the bridge are submerged, the full flowing orifice equation is used. Pressure flow is calculated as orifice flow with the discharge proportional to the square root of the effective head. Submerged orifice flow is treated similarly with the head redefined. HEC-RAS can also simultaneously consider embankment overflow as a weir discharge. The program will begin checking for the possibility of pressure flow when the computed low flow energy grade line is above the maximum low chord elevation at the upstream side of the bridge. Once pressure flow is computed, the pressure flow answer is compared to the low flow answer; the higher of the two is used. The user has the option to tell the program to use the water surface instead of energy to trigger the pressure flow calculation.

Weir Flow Computations

Flow over the bridge, and the roadway approaching the bridge is calculated using the standard weir equation.

$$\mathbf{Q=CLH^{1.5}} \qquad \textbf{(10.1)}$$

The approach velocity is included by using the energy grade line elevation in lieu of the upstream water surface elevation for computing the head, H. The coefficient C, is based on broad crested weir for either free flow conditions (discharge independent of tailwater). For rectangular weir flow over the bridge deck, a coefficient of 2.6 is reasonable. If the weir flow is over the roadway approach, a coefficient of 3.0 is reasonable. If weir flow occurs as a combination of bridge and roadway overflow, then an average coefficient (weighted by weir length) could be used.

For high tailwater elevations, the program will automatically reduce the amount of weir flow to account for submergence on the weir. Submergence is defined as the depth of water above the minimum weir elevation on the downstream side divided by the height of the energy grade line above the minimum weir elevation on the upstream side. The submergence must exceed 75% for any reduction to take place. The reduction of weir flow is accomplished by reducing the weir coefficient based on the amount of submergence. The total weir flow is computed by subdividing the weir crest into segments, computing L, H, a submergence correction, and a Q for each section, them summing the incremental discharges.

When the weir becomes highly submerged the program will automatically switch to calculating the upstream water surface by the energy equation (standard step backwater) instead of using the pressure and weir flow equations. The default submergence is 0.95.

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.2 HEC-RAS Modeling

10.5.2.2.1 General Considerations

HEC-RAS uses some methodology that is different than HEC-2. Studies performed using HEC-2 will result in slightly different water surface profile elevations when recalculated using HEC-RAS. For such sites, the first run should be for the HEC-2 data to establish the base condition prior to making changes that reflect the proposed project.

Several models may need to be developed demonstrating the sequence of events that impact the site. A suggested order is:

A. Existing condition
B. Embankment Encroachment
C. Structure effects

In order to understand the effects of the stream crossing system on the hydraulics of the site, the following discussion is presented. A user's instruction manual for HEC-RAS is available and should be used for information on using the computer model. Two specific examples are given in Appendix B. Only sufficient information to understand the examples is given.

Starting Water Surface

For subcritical flow, it is normal practice to use Manning’s equation to compute normal depth as the starting water surface. The actual water surface may be higher or lower than normal depth. The use of normal depth at the boundary will introduce an error. In general the error at the boundary will diminish as the computations proceed upstream.

The water surface profile used in a subcritical hydraulic analysis should begin far enough downstream, if there is no obvious hydraulic control section, to ensure that the computed water surface elevation has converged to a consistent answer by the downstream limits of the study reach, i.e. reached the “true” depth downstream of the influence of the bridge constriction. The water surface profile shall extend upstream beyond the extent of the bridge backwater caused by the bridge (increase in water surface profile resulting from a channel modification converges with the existing conditions profile).

Conversely, in modeling supercritical flow, the analysis should begin far enough upstream to insure that the water surface elevation has reached the normal depth stage upstream of the influence of the bridge constriction and extends downstream far enough to be beyond the influence of the bridge.

Length of reach

The distances between cross sections are referred to as reach lengths. Cross-sections should be situated such that they represent the channel conditions, and provide for computational stability. Channel reach lengths are typically specified along the thalweg. Overbank reach lengths should be measured along the anticipated path of the center of mass of the overbank flow. In steep channels with shallow flow they should be situated sufficiently close to preclude that the fall exceeds the flow depth. A guide for the spacing of cross-sections is the distance between cross-sections does not exceed 10% of the Average Depth/ Streambed Slope.

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.2.1 General Considerations (continued)

Cross-sections for Modeling

The cross sections that are desired for the energy analysis through the bridge opening for a single opening bridge without spur dikes are shown in Figure 10-1. The additional cross sections that are necessary for computing the entire stream water surface profile are not shown in this figure. Cross sections 1, 2,3, and 4 are required to define the flow approaching and departing the roadway embankment. In addition, sections which define the bridge are needed for the energy loss computations of the bridge structure. The bridge and attendant embankment encroachment should be modeled using 4 sections. There should be an entrance(3) and exit(2) section that describes the embankment encroachment using the geometry of the embankment and bank protections, and bridge sections as appropriate for the upstream(BU) and downstream(BD) faces of the bridge.

Ineffective Flow Modeling

The cross-sections may need to be modified to identify cross section areas that contain water that is not actively conveyed (ineffective flow). The program has the capability to modify cross sections so as to simulate sediment deposition, confine flows to leveed channels, block out road fills and bridge decks, and floodplain encroachments. Ineffective flow areas can be defined using a.) ineffective flow , b.) levees, or c.) blocked areas. As encroachments are changed at a section, other sections must be reviewed for the specification of effective flow areas. Modeled ineffective flow areas may need to be changed with changes in discharge.

Ineffective flow areas are used to describe portions of a cross section in which water will pond, but the velocity of that water, in the downstream direction, is close to zero. The water is included in the storage calculations and other wetted cross section parameters but is not included as part of the active flow area. When using ineffective flow areas, no additional wetted perimeter is added to the active flow area. Ineffective flow areas are modeled by use of levees, or blocked flow. For leveed flow, once the water surface goes above the established elevation, then that specific area is no longer considered ineffective. For blocked flow, the water will not flow within the limits that describe the block, however, a blocked obstruction does not prevent water from going outside of the obstruction. Up to 20 blocks may be entered.

Ineffective Flow at an Encroaching Embankment

Ineffective flow areas are used for the channel sections adjacent to a roadway embankment where there are ponded areas outside the contraction or expansion limits. Use of ineffective flow will allow for considering this area as effective when the roadway is overtopped. The ineffective flow areas should be set at stations that will adequately describe the active flow area at cross section 2 and 3. In general, these stations should be placed outside the edges of the bridge opening to allow for the contraction and expansion of flow that occurs in the immediate vicinity of the bridge. On the upstream side, Section 3, one may use a contraction ratio of 1:1. If section 3 is 10 feet upstream, begin the limits of ineffective flow 10 feet wider than the bridge opening on each side. On the downstream side, Section 2, a similar assumption may be used with an expansion ratio in the range of $0.5 < ER < 4$. See Appendix 10-A for guidance on selecting an expansion ratio.

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.2.1 General Considerations (continued)

The elevations specified for ineffective flow should correspond to elevations where significant weir flow passes over the roadway. This may need to be the elevation at the top of the guard/bridge rail as they are usually blocked with debris. For the downstream cross section, the threshold water surface elevation for weir flow is not usually known for the initial run, it may be estimated using the average elevation between the low chord and the minimum top of road.

The user should check to ensure that the computer solutions are consistent with the type of flow described. For low flow or pressure flow, the output should show the effective areas restricted to the bridge opening. When the output indicates weir flow, the solution should show the entire cross section is effective. The overbank flow around the bridge should be consistent with weir flow.

Levees

Cross sections with low overbank areas that are not part of the main channel require special consideration in computing water surface profiles. Normally the computations are based on the assumption that all area below the water surface elevation is effective in passing the discharge. However, if the water surface elevation at a particular cross section is less than the top of "levee" elevations, and if the water cannot enter or leave the overbanks upstream of that cross section, then the flow areas in these overbanks should not be used in the computations. These areas should only be considered when the flow in the channel is higher than the "levee" elevations. The user excludes these areas by using levees at locations where the water is contained in the main channel until the levee elevation is exceeded. The program includes additional wetted perimeter when water comes in contact with the levee wall. If the water surface elevation is close to the top of a levee, the program may experience some difficulty in balancing the water surface elevations due to changing assumptions of flow area when just above or below the levee top. The designer must review the appropriateness of the assumed water surface elevations and revise the model as necessary. Also, assumptions regarding effective flow areas may change with changes in flow magnitude. Where cross section elevations outside the levee are considerably lower than the channel bottom, this may require adjustment of the cross section to be sure that effective flow areas are properly described.

It is important for the user to study carefully the flow pattern of the river where levees exist. If, for example, a levee were open at both ends and flow passed behind the levee without overtopping it, it should be modeled as a berm rather than described as a "levee".

Blocked Areas

Areas of the cross section from which flow is permanently obstructed are described using blocked area. The areas may be defined using total blocked areas for each side or up to 20 multiple blocks. In either case the blockage is described using stations and elevations of the blocked area.

Energy Losses

Energy losses in HEC-RAS are normally computed using standard step procedures. Friction losses are evaluated using Manning's n values. The accuracy of the computed surface profiles is significantly affected by the appropriateness of the chosen Manning's n. See Chapter 8 for discussion on selection of Manning's n.

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.2.1 General Considerations (continued)

Expansion and Contraction

All models must have the appropriate expansion and contraction encroachment applied at each step. Expansion and contractions are handled by using taper ratios to limit effective flow areas and coefficients. The coefficients are multiplied by the absolute difference in velocity heads between the current cross section and the next cross section downstream. Where the change in river cross section is small, and the flow is subcritical, contraction and expansion coefficients of 0.1 and 0.3 are often used.

Table 3.3 Subcritical Flow Contraction and Expansion Coefficients

	Contraction	Expansion
No transition loss computed	0.0	0.0
Gradual transitions	0.1	0.3
Typical Bridge Sections	0.3	0.5
Abrupt Transitions	0.6	0.8

Flow Distribution Calculations

The program computes the flow distribution based on three flow subdivisions, (left overbank, main channel, and right overbank), and balancing the energy equation. The user can request additional output showing the distribution of flow for multiple subdivisions on each of these areas. The user can select to have this information for specific cross-sections or all cross-sections. Up to 45 slices can be specified. Each flow element (left overbank, main channel, right overbank) must have at least one slice. Where a flow distribution is requested, the program will calculate the flow (discharge), area, wetted perimeter, percentage of conveyance, hydraulic depth, and average velocity for each slice.

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.2. HEC-RAS Modeling (continued)

10.5.2.2.2 Bridge Modeling Approach

Low Flow Methods
For low flow conditions (water surface below the highest point on the low chord of the bridge opening) the Energy and Momentum methods are the most physically based and in general are applicable to the widest range of bridges and flow situations. Both methods account for friction losses and changes in geometry through the bridge. The energy method accounts for additional losses due to flow transitions and turbulence through the use of contraction and expansion losses. The momentum method can account for additional losses due to pier drag.

1. In cases where the bridge piers are a small obstruction to the flow, and fiction losses are the predominate considerations the energy based method or the momentum method should be used.

2. In cases where pier losses and friction losses are predominant, the momentum method is most applicable.

3. Whenever the flow passes through critical depth within the vicinity of the bridge, both the momentum and energy methods are capable of modeling this type of flow transition.

4. For supercritical flow both the energy and the momentum method can be used. The momentum-based method may be better at locations that have a substantial amount of pier impact and drag losses.

5. For bridges in which the piers are the dominant contributor to energy losses and the change in water surface, the momentum method is applicable.

6. For long culverts under low flow conditions, the energy based standard step method is the most suitable approach. However, if the culvert flows full or in inlet control conditions, the culvert routines are the best approach.

High Flow Methods
For high flow (flows that come in contact with the maximum low chord of the bridge deck) the energy based method is applicable to the widest range of conditions.

1. When the bridge deck is a small obstruction to the flow, and the bridge opening is not acting like a pressurized orifice, the energy method should be used.

2. When the bridge deck and road embankment are a large obstruction to the flow, and a backwater created due to the construction of the flow, the pressure and weir method should be used.

3. When the bridge and/or road embankment is overtopped, and the water going over the top of the bridge is not highly submerged by the downstream tailwater, the pressure and weir method should be used. The pressure and weir method will automatically switch to the energy method if the bridge 10.5 Design Procedures & Modeling Considerations (continued)

10.5.2 Water Surface Profile Analysis at Bridges (continued)

10.5.2.2.2 Bridge Modeling Approach (continued)

becomes 95 percent submerged. The user can change the percent submergence at witch the program switches from the pressure and weir method to the energy method.

4. When the bridge is highly submerged, and flow over the road is not acting like weir flow, the energy based method should be used.

5. When describing the bridge superstructure for overtopping, include the guardrail and bridge rail.

Unique Challenges and Suggested Approaches

Perched Bridge

A perched bridge is one for which the road approaching the bridge is at the floodplain ground level and in the immediate vicinity of the bridge the road rises above the ground level to span the watercourse. A typical low-flow situation is low flow under the bridge and overbank flow around the bridge. If the road approaching the bridge is not much higher than the surrounding ground, the assumption of weir flow is not justified. A solution based on the energy method is a better solution, especially when a large percentage of the total discharge is in the overbank areas.

High Submergence Bridge (Low flow bridge)

A low flow bridge is designed to carry only low flows under the bridge. Flood flows are carried over the bridge and road. When modeling this condition for flood flows, the solution may be by either a combination of pressure and weir flow method or energy flow method. If the tailwater is high, it may be better to use the energy-based method.

Bridges on a Skew

Skewed bridge crossings are generally handled by making adjustments to the bridge dimensions to define an equivalent cross section perpendicular to the flow lines. Skewed crossing with angles up to 20 degrees show no objectionable flow patterns.

Multiple Openings

Multiple openings of either bridge and/or culvert can be modeled using HEC-RAS. One should consider using flow distribution output to review the hydraulic modeling.

Multiple/Parallel Bridges

The hydraulic loss through multiple bridges is between one to two times for one bridge. If the bridges are far enough apart, the loss for the multiple bridges is equal to sum of the losses for each bridge. If the bridges are very close together and the flow is not able to expand between the bridges, the contraction should occur only at the upstream bridge with only pier and friction losses at the downstream bridges, the bridges can be modeled as a single wide bridge. If there is sufficient distance between the bridges in which the flow has room to expand and contract, the bridges should be modeled as separate bridges. If separate bridges are modeled, the expansion and contraction rates should be based on the same procedure as for a single bridge.

10.6 Bridge Scour

10.6.1 Introduction--Philosophy

Reasonable and prudent hydraulic analysis of a bridge design requires that an assessment be made of the proposed bridge's vulnerability to undermining due to potential scour.

It is ADOT's Goal that the bridge does not fail during its lifetime due to scour for flow events up-to and including the 500-year event. In this context, lifetime means the period of time that the bridge is in use. It is not limited to the economic life or the design life of the highway.

10.6.2 Scour Types

Scour is the manifestation of the sediment transport process as affected by the presence of objects that change or disturb the approach flow conditions. The object that causes disturbance can be a bend in the bank, an encroaching embankment or a pier. Any of these objects causes a change in the sediment transport capacity of the flow, and a corresponding scour.

Bridge scour is evaluated as interrelated components:

- long term profile changes (aggradation/degradation),
- plan form change (lateral channel movement, bank widening),
- contraction scour/deposition, and
- local scour.

Presented are the generally recommended procedures for use in most situations. In line with the goal presented above, the designer is encouraged to apply all the information available and make judgments that **ensure the bridge will NOT fail**.

Long Term Profile Changes

Aggradation or degradation long-term profile changes can result from streambed gradient or sediment transport changes.

- Aggradation is the deposition of bedload due to a decrease in the local stream sediment transport capacity. This may be due to a decrease in the local energy gradient.

- Degradation is the scouring of bed material due to an increase in the local stream sediment transport capacity. This may result from an increase in the energy gradient or from removal of sediment at an upstream section due to gravel mining or a reservoir.

10.6 Bridge Scour (continued)

Long Term Profile Changes (continued)

Where gravel mining is an on-going or to be expected activity, the impacts of the change in the sediment transported may result in long-term profile changes.

- Gravel mining can cause an increase in the energy gradient at the upstream end of the operation that may result in degradation of the streambed profile.

- Gravel mining can reduce the sediment transported to downstream reaches that may result in degradation of the streambed profile.

Plan Form Changes

Plan form changes are morphological changes such as meander migration or bank widening. The lateral movement of meanders can threaten bridge approaches as well as increase scour by changing flow patterns approaching a bridge opening. Bank widening can cause significant changes in the flow distribution and thus the bridge's flow contraction ratio.

Constriction & Expansion

Contraction scour results from a constriction of the flow area that may, in part, be caused by bridge piers in the waterway in addition to the encroaching embankment/abutment. Deposition results from an expansion of the channel or the bridge site being positioned immediately at the beginning of a flatter reach of stream.

Highways, bridges, and natural channel constrictions are the most commonly encountered cause of contraction scour. The scour is considered as either live-bed or clear water contraction scour. Live-bed scour occurs when bed material is already being transported into the contracted opening from upstream of the approach section. Clear water contraction scour occurs when the bed material transport in the uncontracted approach section is negligible or less than the carrying capacity of the flow. The two most common occurrences of contraction scour for bridges in Arizona are

Case I. Overbank flow on a floodplain being forced back into the main channel. Case I occurrences include the following conditions:

a. The river channel width becomes narrower either due to the bridge abutments projecting into the channel or the bridge being located at a narrowing reach of the river.

b. Overbank flow area is obstructed either partially or completely by the road embankment/abutment with no constriction of the main channel

Case II. Flow is confined to the main channel (i.e. there is no overbank flow). The normal river channel becomes narrower due to the bridge itself or the bridge site is located at a narrowing reach of the river.

10.6 Bridge Scour (continued)

10.6.2 Scour Types (continued)

Local Scour

Exacerbating the potential scour hazard at a bridge site are any abutments or piers located within the flood flow prism. The amount of potential scour caused by these features is termed local scour. Local scour is a function of the geometry of these features as they relate to the flow geometry.

10.6.3 Armoring & Scour Resistant Materials

Armoring

Armoring occurs because a stream or river is unable, during a particular flood event, to move the more coarse material comprising either the bed or, if some bed scour occurs, its underlying material. A review of the armored material may reveal well-rounded material that has been transported; not a coarse resident bed material. Scour may occur initially but later become arrested by armoring before the full scour potential is reached again for a given flood magnitude. When armoring does occur, the coarser bed material will tend to remain in place or quickly redeposit so as to form a layer of riprap-like armor on the streambed or in the scour holes and thus limit further scour for a particular discharge. When a larger flood occurs than the flood that created the armoring, scour will probably penetrate deeper until armoring again occurs at some deeper threshold.

Armoring may result in the stream being unable to satisfy it desired sediment transport capacity, this may cause bank widening. Bank widening encourages rivers or streams to seek a more unstable, braided regime. Such instabilities may pose serious problems for bridges as they encourage further, difficult to assess plan form changes. Bank widening also spreads the approach flow distribution that in turn results in a more severe bridge opening contraction.

Scour Resistant Materials

Caution is necessary in determining the scour resistance of bed materials and the underlying strata. Serious scour has been observed to occur in materials commonly perceived to be scour resistant such as consolidated soils and glacial till, as well as so-called bedrock streams and streams with gravel and boulder beds. With sand size material, the passage of a single flood may result in the predicted scour depths. Conversely, in scour resistant material the maximum predicted depth of scour might not be realized during the passage of a particular flood; however, some scour resistant material may be lost. Commonly, the stream will replace this material with transported material that is more easily scoured. Thus, at some later date another flood may reach the predicted scour depth.

10.6.4 Pressure Flow Scour

Pressure flow, which is also denoted as orifice flow, results from a pile up of water on the upstream bridge face, and a plunging of the flow downward and under the bridge. This occurs when the water surface elevation at the upstream face of the bridge is greater than or equal to the low chord of the bridge superstructure. At higher approach flow depths, the bridge can be entirely submerged with the resulting flow being a complex combination of the plunging flow under the bridge and the flow over the bridge.

With pressure flow, the local scour depths at a pier or abutment are larger than for free surface flow with similar depths and approach velocities. The increase in local scour at a pier subject to pressure flow results from the flow being directed downwards toward the bed by the superstructure and by increasing the

10.6 Bridge Scour (continued)

10.6.4 Pressure Flow Scour (continued)

intensity of the horseshoe vortex. The vertical contraction of the flow is a more significant cause of the increase in scour depth. However, in many cases, when a bridge becomes submerged, the average velocity under it is reduced due to a combination of additional backwater caused by the bridge superstructure impeding the flow and a reduction of discharge which must pass under the bridge due to weir flow over the bridge and approach embankments. As a consequence of this, increases in local scour attributed to pressure flow scour at a particular site, may be offset to a degree by lesser velocities through the bridge opening due to increased backwater and a reduction in discharge under the bridge due to overtopping.

HEC-RAS can be used to determine the discharge through the bridge and the velocity of approach and depth upstream of the piers when flow impacts the bridge superstructure. These values should be used to calculate local pier scour. Engineering judgment will then be exercised to determine the appropriate multiplier times the calculated pier scour depth for the pressure flow scour depth. This multiplier ranges from 1.0 for a low approach Froude numbers (Fr = 0.1) to 1.6 for high approach Froude numbers (Fr = 0.6). If the bridge is over- topped, the depth to be used in the pier scour equations and for computing the Froude number is the depth to the top of the bridge deck or guardrail obstructing the flow.

10.6.5 Scour Prediction Methodology

Bridge scour assessment shall be accomplished by collecting the data and applying the general procedure outlined in this section.

10.6.5.1 Site Data

Geometry

Obtain existing stream and flood plain cross sections and profile, site plan and the stream's present, and where possible, historic geomorphic plan and profile form. Also, locate the bridge site with respect to such things as other bridges in the area, tributaries to the stream or close to the site, bedrock controls, manmade controls (dams, old check structures, river training works, etc.), and downstream confluence with other streams. Locate (distance and height) any "headcuts" due to natural causes or activities such as gravel mining operations. Data related to plan form changes such as meander migration and the rate at which they may be occurring are useful.

When gravel mining is an on-going activity or should be expected, the impacts of gravel mining on the stream shall be evaluated. Upstream gravel mining operations may “capture” the bed material discharge resulting in the more adverse clear water scour case discussed later. Current practice is to include an allowance for future degradation at the bridge site where extensive mining is occurring, such as the Salt River in Phoenix or the Santa Cruz River in Tucson. For isolated gravel mining, an estimate of the degradation depth may be made using the procedures in “Gravel Mining Guidelines” reference “Effects of In-stream Mining on Channel Stability”

10.6 Bridge Scour (continued)

10.6.5.1 Site Data (continued)

Bed Material

The bed material should be observed. Look for evidence of bedrock outcrops, grain size for determination of bed forms and type of scour, and other indicators of the morphology of the stream. It is ADOT practice to consider all material to be scour susceptible unless proven otherwise. Grain size is not considered for the purpose of decreasing the predicted scour.

Geomorphology

Classify the geomorphology of the site; i.e., such things as whether it is a flood plain stream or crosses an alluvial fan; youthful, mature or old age, presence of headcuts, and meanders.

Historic Scour

Review available information such as as-builts, bridge inspection reports, old contour mapping, and aerial photographs to evaluate scour data on other bridges or similar facilities along the stream.

Debris

A build up of debris on the pier shall be considered. In the absence of additional site information, the **debris shall be assumed to extend 2 feet on each side of the pier and have a depth of 12 feet from the water surface**.

Hydrology

Identify the character of the stream hydrology; i.e., perennial, ephemeral, intermittent as well as whether it is "flashy" or subject to broad hydrograph peaks resulting from gradual flow increases such as occur with general thunderstorms or snowmelt and dam releases. The operational design frequency flood, the superflood and the greatest discharge passing through the structure, if less than the superflood, will be required.

10.6.5.2 Scour Prediction Practice

It is ADOT's practice to calculate the scour components as if they develop independently. Thus, the potential local scour is added to the contraction scour without considering the effects of contraction scour on the channel and bridge hydraulics. The general approach with this method is as follows. No reduction in the predicted scour is considered for armoring.

- The ADOT bridge manual specifies different load cases for analysis of the bridge capacity. These load cases must be considered in calculating the predicted scour.
- Estimate the natural channel's hydraulics for a fixed bed condition based on existing conditions.
- Assess the expected profile and plan form changes.

10.6 Bridge Scour (continued)

10.6.5. 2 Scour Prediction Practice (continued)

- Adjust the fixed bed hydraulics to reflect any expected profile or plan form changes.

- Estimate contraction scour using the empirical contraction formula.

- Estimate local scour using the channel and bridge hydraulics assuming no bed armoring. Abutment scour is estimated using Froelich's method as demonstrated in Appendix D. Pier scour is estimated using the CSU equations as demonstrated in Appendix E.

- Add the contraction scour to the local pier scour to obtain the total scour. The Abutment scour predicted includes the Contraction scour.

- The resultant predicted scour needs to be discussed with the geotechnical and bridge designers to confirm the assumptions made and to verify the erodibility of the bed material. As the goal is for the bridge not to fail; that the bed material is non-erodible is the case that must be proven.

10.6.5.3 General Procedure

Step 1

Determine the magnitude of the design flood and the superflood as well as the magnitude of the incipient overtopping flood, or relief-opening flood. Accomplish steps 2 through 10 using the discharges computed.

Review the site to identify the discharge that places the greatest stress on the bed material in the bridge opening. Where there are relief structures on the flood plain or overtopping occurs, some flood other than the base flood or "super flood" may cause the worse case bridge opening scour. This situation occurs where the bridge opening will pass the greatest discharge just prior to incurring a discharge relief from overtopping or a flood plain relief opening. Conversely care must be exercised in that a discharge relief at the bridge due to overtopping or relief openings might not result in reduction in the bridge opening discharge. Should a reduction in the bridge opening discharge occur, the incipient overtopping flood or the overtopping flood corresponding to the base flood or "super flood" is to be used to evaluate the bridge scour.

Step 2

Assess the bridge crossing reach of the stream for <u>profile bed</u> scour changes to be expected from degradation or aggradation. Take into account past, present and future conditions of the stream and watershed in order to forecast what the elevation of the bed might be in the future. Certain plan form changes such as migrating meanders causing channel cutoffs would be important in assessing future streambed profile elevations. The possibility of downstream mining operations inducing "headcuts" shall be considered. The quickest way to assess streambed elevation changes due to "headcuts" (degradation) is by obtaining a vertical measurement of the downstream "headcut(s)" and projecting that measurement(s) to the bridge site using the existing stream slopes if it is acceptable to assume the stream is in regime conditions; if it is not, then it may be necessary to estimate the regime slope. A more time consuming way to assess elevation changes would be to use a sediment routing practice in conjunction with a synthetic flood history.

10.6 Bridge Scour (continued)

10.6.5.3 General Procedure (continued)

Step 3

Assess the bridge crossing reach of the stream for plan form scour changes. Attempt to forecast whether an encroaching meander will cause future problems within the expected life of the bridge. Take into account past, present and expected future conditions of the stream and watershed in order to forecast how such meanders might influence the approach flow direction in the future. The sediment routing practice discussed later for computing channel contraction scour or aggradation may prove useful in making such assessments. This forensic analysis on a site's past geomorphological history may prove useful to forecast the future. Otherwise this assessment has to be largely subjective in nature.

Step 4

Develop a water surface profile for the discharges to be considered through the site's reach for fixed bed conditions using HEC-RAS.

Step 5

Assess the magnitude of contraction scour based on the fixed bed hydraulics.

Step 6

Assess the magnitude of local scour at abutments and piers. See section 10.6.5.4 and 10.6.5.5.

Step 7

For each discharge under consideration, plot the scour and aggradation depths from foregoing steps on a cross section of the stream channel and flood plain at the bridge site. Using judgment, enlarge any overlapping scour holes (discussed later).

Step 8

Discuss the predicted condition with the bridge and geotechnical design staff.

10.6.5.4 Contraction Scour

Contraction scour is an application of the principle of conservation of sediment transport. Live-bed contraction scour occurs at a bridge when there is transport of bed material in the upstream reach in to the bridge cross-section. In live-bed scour, the area of the contracted section increases until the fully developed scour in the bridge cross-section reaches equilibrium when sediment transported into the contracted section equals sediment transported out. As scour develops, the shear stress in the contracted section decreases as a result of a larger flow area and decreasing average velocity. For live-bed scour, maximum scour occurs when the shear stress reduces to the point that sediment transported in equals the bed sediment transported out and the conditions for sediment continuity are in balance.

10.6 Bridge Scour (continued)

10.6.5.4 Contraction Scour (continued)

Clear water contraction scour occurs when (1) there is no bed material transport from the upstream reach into the downstream reach, or (2) the material being transported in the upstream reach is transported through the downstream reach mostly in suspension and at less than the capacity of the flow. For clear-water scour, the transport into the contracted section is essentially zero and maximum scour occurs when the shear stress reduces to the critical shear stress of the bed material in the section. With clear-water contraction scour the area of the contracted section increases until, in the limit, the velocity of the flow (v) or the shear stress (t_c) on the bed is equal to the critical velocity (v_c) or the critical shear stress (t_c) for the representative particle size D, in the bed material.

There are four cases of contraction scour at bridge sites depending on the type of contraction, and whether there is overbank flow or relief bridges. The two most common occurrences of contraction scour for bridges in Arizona are presented on page 10-31. For any condition, it is only necessary to determine if the flow is transporting bed material (live-bed) or is not (clear water), and then apply the equation appropriate for the case with the variables defined according to the location of contraction scour (channel or overbank).

To determine if the flow upstream of the bridge is transporting bed material, calculate the critical velocity for beginning of motion V_c of the D50 size of the bed material being considered for movement and compare with the mean velocity of the flow in the main channel or the overbank area upstream of the bridge opening. If the critical velocity of the bed material is larger than the mean velocity ($V_c > V_0$), then clear water scour will exist. If the critical velocity is less than the mean velocity ($V_c < V_0$), then live-bed contraction scour will exist. The equation for the critical velocity is

$$V_c = 11.17\, y^{1/6} D_{50}^{1/3} \qquad \textbf{(10.2)}$$

Where:

V_c = Critical velocity above which bed material of size D and smaller will be transported, ft/sec

$y^{1/6}$= Average depth of flow upstream of the bridge, ft

$D_{50}^{1/3}$= Particle size in a mixture of which 50 percent are smaller, ft.

The D_{50} is taken as an average of the bed material size in the reach of the stream upstream of the bridge. It is a characteristic size of the material that will be transported by the stream. Normally this would be the bed material in the surface 1 foot of the streambed.

10.6 Bridge Scour (continued)

10.6.5 Scour Prediction methods (continued)

10.6.5.4 Contraction Scour (continued)

Laursen (1980) in **Predicting Scour at Bridge Piers and Abutments** made the following comments:

- Scour is first of all a result of the nonuniform flow pattern whereby the capacity (or competence) for sediment movement is different in one area than in the immediately upstream area. Therefore, scour is first of all a function of geometry which determines flow pattern

- Scour at abutments is the same as scour in a long contraction in that it is a consequence of an imbalance of the supply of sediment to an area and the capacity to move sediment out of that area.

- The limit of scour at an abutment is either a balance of supply and capacity (scour by sediment-transporting flow) or a boundary shear equal to the critical tractive force (clear-water scour).

- Abutment scour is caused by the flow that is forced to be turned by the embankment. The scour hole occurs within the width of the obstructed flow at the point of joining the main flow.

On page 57 Laursen makes these observations:

- The flow and sediment transport in the channel beyond the lateral limits of a scour hole are not noticeably affected by the obstruction or the scour hole.

- The flow approaching a scour hole, but outside the lateral limits of the actual obstruction, is virtually unaffected by the obstruction although the sediment being transported, of course, falls into the hole.

- The flow that approaches the obstruction dives down into the scour hole, takes the form of a spiral roller within the scour hole which bends around the sides of the obstruction and out of the scour hole in a flat tail as it mixes with the flow above. The spiral roller is the agent which moves the sediment on out of the scour hole.

There are many procedures available for computing predicted abutment scour. ADOT recommends the use of Froehlich's equation for calculating abutment scour as described below. The predicted scour should be checked with the HIRE equation.

Froehlich's equation for calculation of predicted abutment scour is

$$y_{s}/\, y_{a} = 2.27 * K_{1} * K_{2} * (L'/y_{a})^{0.43} * Fr^{0.61} + 1 \qquad \textbf{(10.3)}$$

where:

Y_S= predicted abutment scour, ft.
y_a = Average flow depth on the floodplain, ft.
L' = Length of active flow obstructed by the abutment and approach embankment, ft
Fr = Froude number based on the velocity just upstream of the abutment.

10.6 Bridge Scour (continued)

10.6.5.4 Abutment Scour (continued)

Factor Values:

K_1= factor for abutment shape.
K_1= 1.0, vertical wall abutment, to be used when evaluating the capacity of the abutment foundation with an eroded embankment slope.
K_1= 0.55, spill through abutment, can be used for the design depth of bank protection when failure of the bank protection does not endanger the safety of the bridge.

K_2= factor for angle between flow direction and embankment alignment.
$K_2 = (Q/90)^{0.13}$

$Q<90$ if embankment points downstream
$Q>90$ if embankment points upstream

Based on the bridge crossing layout, determine the right and left encroachment lengths. As mentioned in HEC-18, abutment scour depends on the interaction of the flow obstructed by the abutment and roadway approach and the flow in the main channel. To apply these equations, the obstructed flow and the approach flow must be determined. Where Q_{ob} is the obstructed flow and q_w is the unit flow adjacent to the abutment.

y_a = Average flow depth on the floodplain, ft.
$y_a = A_{ob}/T_{ob}$, where A_{ob} is the area of the obstructed flow and T_{ob} is the top width for the obstructed flow

HIRE: For HIRE, scour is a function of the depth and Froude number for the flow adjacent to the abutment.

$y_s = 4.0*K_1/0.55*K_2 *Fr^{0.33}\ y_a$

with K_1, K_2, and y_a the same as for Froelich.

The abutment scour prediction should follow the following steps:

1. Run a hydraulic model to determine the main channel, left and right overbank discharges
2. Determine Q_o, q_w, and A_{ob}.
3. Determine $L' = Q_{ob}/q_w$ and $y_a = A_{ob}/T_{ob}$.
4. Apply the values for L' and y_a in equation 10.3. ADOT makes the following adjustment in the application of equation 10.3. **If L'/y_a is greater than 25, set L'/y_a to 25 and solve for Y_s. Compare with the scour resulting from use of HIRE. Use the larger scour value.**

10.6 Bridge Scour (continued)

10.6.5.4 Abutment Scour (continued)

5. The datum for the predicted depth is the channel bottom without contraction scour. The predicted depth includes both abutment scour and contraction scour.

See Appendix D for an example and comparison of abutment scour computations.

10.6.5.5 Pier Scour

At bridge piers, the flow that is obstructed changes into a “horseshoe” pattern around the pier. There is also a component that flows downward along the pier. This flow continues downward until either the resistance of the water consumes the energy or another obstruction is encountered. If the obstruction is the bed of the stream, the flow will, depending on the grain size, loosen the bed material creating a hole. The flow will carry the displaced material out of the hole until the combination of grain size, stream force, and depth of hole result in an equilibrium condition with no additional material being carried out of the hole for clear water scour, or a balance of sediment inflow and outflow transport for live-bed scour.

The scour is related to the affected discharge, ($a*Y_o$), and the relative velocity of flow.

The recommended equation for calculation of predicted pier scour is

$$(Y_s/Y_o)=2.0* K_1*K_2*K_3*K_4*(a/Y_o)^{0.65}*Fr^{0.43} \qquad \textbf{(10.4)}$$

where:

Y_s= predicted pier scour, ft.

Y_o=flow depth upstream of pier, ft.

a = pier width, including adjustment for multiple piers and debris, ft

Fr=Froude number based on the velocity just upstream of the pier.

Factor Values:

K_1= factor for pier nose shape.

K_1=1.1, ADOT policy is that the pier shape is affected by debris, consider K_1 as 1.1.

K_2= factor for angle between flow direction and pier alignment.

Use the projected width of the pier, determined from $K_2=(\cos \Omega+(L/a)*\sin \Omega)^{0.65}$

with a limiting value of 5.0. See Table 10.6.1.

K_3= factor for bed condition.

K_3= 1.1 for most cases where plane bed conditions exist, dune height <= 10 feet.

K_4=factor for armoring by bed material.

K_4 = 1.0, ADOT policy is for no decrease due to bed armoring.

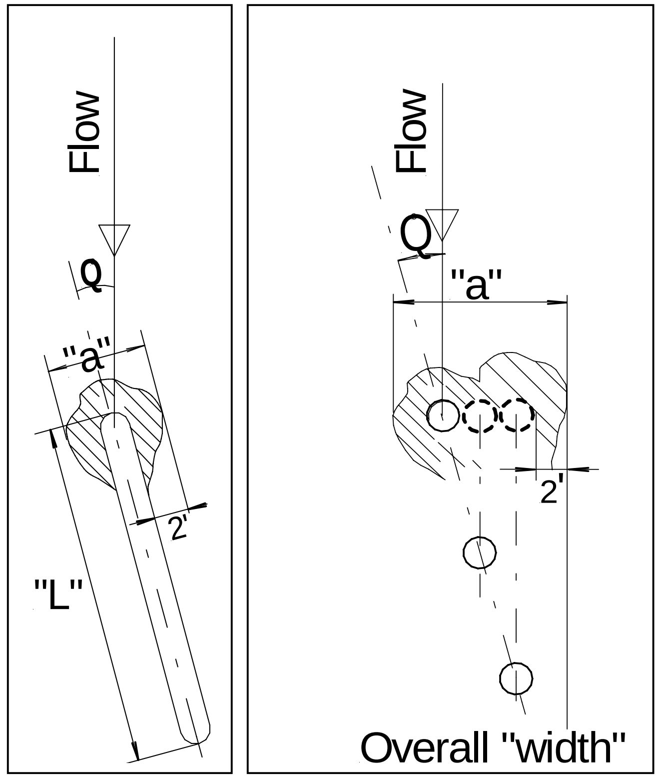
For these equations, it is assumed that the bed material is sufficiently fine-grained so that the material gradation does not affect the predicted scour.

10.6 Bridge Scour (continued)

10.6.5.5 Pier Scour (continued)

Table 10.6.1
Correction factor, K_2
for angle of attack of the flow
$K_2=(\cos(\theta)+(L/a)*\sin(\theta))^{0.65}$

Angle	L/a=4	L/a=8	L/a=12
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0



Where
θ= skew angle of flow,
ADOT requires a minimum value of 15 degrees,
site conditions may require a greater value.
L = length of pier, ft
a = effective width of pier including debris, ft.
If L/a is greater than 12, use the values for L/a =12.

In the absence of additional site information, for all piers, a debris width of 4 feet will be added to the normal width of pier. The debris shall be assumed to extend to a depth of 12 feet from the water surface or flow depth, whichever is less.

For pier frame with multiple columns, if the clear opening width between columns is greater than 16 feet, they shall be evaluated as single columns. If the clear opening width between columns is less than 16 feet, the overall width over the exterior columns shall be used as the obstructed width, a. Debris width shall be included in the width determined above. For this condition, use K_2=1.

Piers that are located close to an abutment, (such as at the toe of a spill through abutment), must be carefully evaluated. They may be exposed to a much higher angle of attack and velocity. Also they may be within the influence of the predicted “abutment scour”.

See Appendix E for an example of pier scour computation.

10.7 References

AASHTO, Volume VII-Highway Drainage Guidelines, "Hydraulic Analyses for the Location and Design of Bridges", AASHTO Task Force on Hydrology and Hydraulics, 1982.

Arizona Department of Transportation, “Effects of In-stream Mining on Channel Stability”, Report Number: FHWA AZ89-250

Bradley, J.N., "Hydraulics of Bridge Waterways," HDS-1, Federal Highway Administration, 1978.

Corry, M.L., Jones, J.S., and Thompson, P.L., "The Design of Encroachments on Flood Plains Using Risk Analysis, "Hydraulic Engineering Circular No. 17, Federal Highway Administration, Washington, D.C., 1980.

Federal Highway Administration, "Highways in the River Environment-Hydraulic and Environmental Design Considerations", Training and Design Manual, Federal Highway Administration, 1975.

Federal Highway Administration, "Federal Highway Program Manual," Vol. 6, Ch. 7, Sec. 3, Subsec. 2, November 1979.

Federal Highway Administration "Evaluating Scour at Bridges", Fourth Edition. HEC-18, 2001.

Kindsvater, C.E., "Discharge Characteristics of Embankment-Shaped Weirs, "U.S. Geological Survey, WSP 1607-A, 1964.

Laursen, E.M., “Predicting Scour at Bridge Piers and Abutments”, University of Arizona, February 1980.

Matthai, H.F., "Measurement of Peak Discharge at Width Contractions by Indirect Methods, "U.S. Geological Survey, Techniques of Water Resources Investigations, Book 3, Ch. A4, 1967.

Schneider, V.R., Board, J.W., Colson, B.E., Lee, F.N., and Druffel, L., "Computation of Backwater and Discharge at Width Constriction of Heavily Vegetated Flood Plains, "U.S. Geological Survey, WRI 76-129, 1977.

Shearman, J.O., "WSPRO User's Instructions", Draft Copy, U.S. Geological Survey, July 1987.

Traille, L.A., and Cherry, D.L., “Flow Modifications by Storage Loss Through Flood Plain Encroachments”, National Cooperative Highway Research Program, NCHRP project 15-7, August 1983

U.S. Army Corps of Engineers, "HEC-2 Water Surface Profiles," User's Manual, September 1982.

U.S. Army Corps of Engineers, "Accuracy of Computed Water Surface Profiles", December 1986.

U.S. Army Corps of Engineers, "HEC-RAS River Analysis System" User's Manual, January 2001.

Appendix A Flow Transitions in Bridge Backwater Analysis

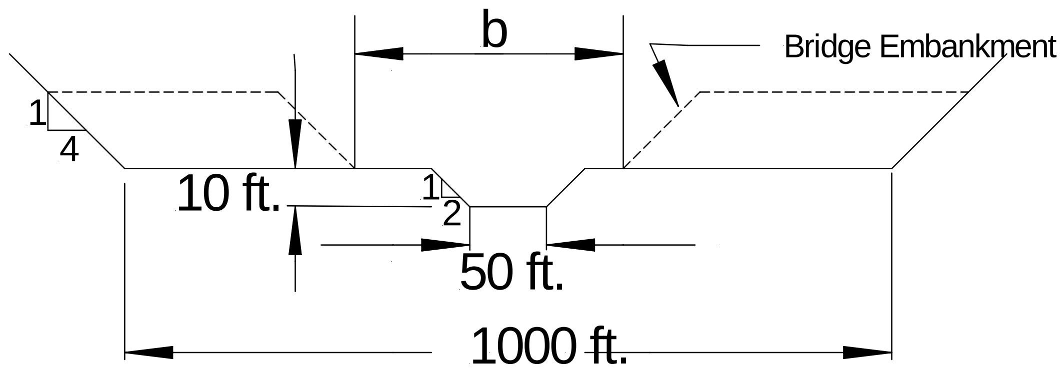
Bridges across floodplains, if they cause severe contraction and expansion of the flow, require special attention in one-dimensional modeling. The accurate prediction of the energy losses in the contraction reach and expansion reach requires the accurate evaluation of four parameters: the expansion reach length, L_e, the contraction reach length, L_c; the expansion coefficient, C_e; and the contraction coefficient, C_c. Research was conducted to investigate these four parameters through the use of field data, two-dimensional modeling, and one-dimensional modeling. The data consisted of 3 actual bridge sites and 76 idealized bridge sites. The field data had the following characteristics: wide, heavily vegetated overbanks, with Manning's n values from 0.07 to 0.24, and slopes between 2.5 feet/mile and 8.0 feet/mile. The idealized bridge sites used the following data:

B=1000 Feet
Bridge opening: 100, 250, and 500 feet
Discharge: 5000, 10000, 20000, and 30000 cfs

Main channel Manning's n = 0.04
Overbank Manning's n: 0.04, 0.08, and 0.16.

Bed slope: 1,5, and 10 feet/mile.

The idealized cross section had the following shape:



The summary statistics for the idealized cases had the following results:

Table 10.A.2
Summary Statistics

Variable	L_e	L_c	C_e	C_c
Sample Size	76	76	76	76
Average	564	386	0.27	0.44
Median	500	360	0.30	0.10
Standard Deviation	249	86	0.15	0.06
Minimum	260	275	0.10	0.10
Maximum	1600	655	0.65	0.50
Range	1340	380	0.55	0.40

Appendix A Flow Transitions in Bridge Backwater Analysis (continued)

CONCLUSIONS

Expansion Reach Length (L_e)

None of the two dimensional cases created for the study had an expansion ratio as great at 4:1. Most of the cases had expansion ratios between 1:1 and 2:1. The use of a 4:1 leads to a consistent over prediction of the energy losses in the expansion reach in most cases. The accompanying over prediction of the water surface elevation at the downstream face of the bridge may be conservative for flood stage prediction studies. For bridge scour studies, this overestimation of the tailwater elevation could in some circumstances lead to an underestimation of the scour potential.

It was found that the ratio of the channel Froude number at Section 2 to that at Section 1, (F_{c2}/F_{c1}), correlated strongly with the length of the expansion reach. Regression equations were developed for both the expansion reach length and ratio. The equation for expansion length also includes the average obstruction length. To use these regression equations in the application of one-dimensional model will usually require an iterative process since the hydraulic properties at section 2 will not be know in advance.

The value of the Froude number ratio reveals important information about the relationship between the constricted flow and the normal flow conditions. It is in effect a measure of the degree of flow constriction since it compares the intensity of flow at the two locations. Since these Froude numbers are for the main channel only, the value of F_{c1} also happens to reflect to some extent the distribution of flow between the overbanks and main channel.

Expansion Coefficients

Regression analysis for this parameter was only marginally successful. The resulting relationship is a function of the ration of hydraulic depth in the overbank to than in the main channel for undisturbed conditions (evaluated at Section 1).

Contraction reach length (L_c)

The contraction ratios for the idealized cases ranged from 0.7:1 to 2.3:1. As with the expansion reach length, these values correlated strongly with the same Froude number ratio. A more important independent variable, however, is the decimal fraction of the total discharge conveyed in the overbanks (Q_{ob}/Q) at the approach section. A strong regression equation was developed for the contraction length.

Contraction Coefficients

69 out of 76 cases had the minimum value of 0.10, making a regression analysis not fruitful.

Asymmetric Bridge Openings

Six idealized cases were developed which had asymmetric bridge obstructions. For these data, the averages of the reach length for the two corresponding symmetric cases closely approximated the values determined for the asymmetric cases. When the regression equations for L_e, ER and L_c were applied to the asymmetric cases, the predicted values were near the observed values.

Appendix A Flow Transitions in Bridge Backwater Analysis (continued)

RECOMMENDATIONS

In applying these recommendations, the modeler should always consider the range of hydraulic and geometric conditions included in the data.

Expansion reach length

Table B.2 offers ranges of expansion ratios that can be used for different degrees of constriction, slopes, and different ratios of overbank roughness to main channel roughness. Once an expansion ratio is selected, the distance to the downstream end of the expansion reach (distance L_e) is found by multiplying the expansion ratio by the average obstruction length (the average of the distances A to B and C to D) The average obstruction length is half of the total reduction in the floodplain width caused by the two bridge approach embankments. In table B.2, for each range, the higher the value is associated with a higher discharge. The ranges in Table B.2 capture the ranges of the idealized model data.

Table 10.A.2
Expansion Ratio

"b/B"	S, ft/mile	n_{ob}/n_{mc}=1	n_{ob}/n_{mc} =2	n_{ob}/n_{mc} =4
b/B=0.1	1 ft/mile	1.4-3.6	1.3-3.0	1.2-2.1
	5 ft/mile	1.0-2.5	0.8-2.0	0.8-2.0
	10 ft/mile	1.0-2.2	0.8-2.0	0.8-2.0
b/B=0.25	1 ft/mile	1.6-3.0	1.4-2.5	1.2-2.0
	5 ft/mile	1.5-2.5	1.3-2.0	1.3-2.0
	10 ft./mile	1.5-2.0	1.3-2.0	1.3-2.0
b/B=0.50	1 ft/mile	1.4-2.6	1.3-2.9	1.2-1.4
	5 ft/mile	1.3-2.1	1.2-1.6	1.0-1.4
	10 ft./mile	1.3-2.0	1.2-1.5	1.0-1.4

The b/B is the ratio bridge opening width to total floodplain width, n_{ob} is the Manning n value for the overbank, n_{mc} is the Manning n value for the main channel and S is the longitudinal slope.

Extrapolation of expansion ratios, slopes or roughness rations outside the range used in this table should be done with care. The expansion ratio should not exceed 4:1, nor should it be less than 0.5:1. The data used to develop the recommendations had a main channel n value of 0.04. For significantly higher or lower main channel n values, the n value ratio will have a different meaning with respect to the overbank roughness.

The regression equation for the expansion reach length is as follows:

$$L_e = -298 + 257\,(F_{c2}/F_{c1}) + 0.918\,L_{obs} + 0.00479Q \qquad \textbf{(10.A.1)}$$

Appendix A Flow Transitions in Bridge Backwater Analysis (continued)

Expansion reach length (continued)

Where : L_e = length of expansion reach, in feet
F_{c2} = main channel Froude number at Section 2
F_{c1} = main channel Froude number at Section 1
L_{obs} = average length of obstruction caused by the two bridge approaches, in feet, and
Q = total discharge, cfs.

When the width of the floodplain and the discharge are smaller than those of the regression data (100 ft wide and 5000 cfs discharge) the expansion ratio can be estimated by equation 10.A.2.

$$ER = L_e/L_{obs} = 0.421 + 0.485*(F_{c2}/F_{c1}) + 0.000\,018\,Q \qquad \textbf{(10.A.2)}$$

When the scale of the floodplain is significantly larger than that of the data, particularly when the discharge is much higher than 30,000 cfs, Equation 10.A.1 and 10.A 2 will over estimate the expansion reach length. Equation 10.A.3 should be used:

$$ER = L_e/L_{obs} = 0.489 + 0.608*(F_{c2}/F_{c1}) \qquad \textbf{(10.A.3)}$$

The depth at Section 2 is dependent upon the reach length, and the Froude number at the same section is a function of the depth. This means that an iterative process is required to use the three equations above. It is recommended that the user start with an expansion ratio from Table 10.A.1, locate section 1 according to that expansion ratio, set the main channel and overbank reach lengths as appropriate, and limit the effective flow area at section 2 to the approximate bridge opening width. The program should then be run and the main channel Froude number values at Section 2 and Section 1 read from the model output. Use these Froude numbers to determine a new expansion length from the appropriate equations, move section 1 and re-compute. When the expansion ratio is large, the resulting reach length may require intermediate cross sections, which reflect the changing width of the effective flow area.

Expansion Coefficient

The analysis of the data with regard to the expansion coefficients did not yield a regression equation that fit the data will. Equation 10.A.6 was the best equation obtained. It is recommended that the modeler use equation 10.A.6 to find an initial value, then perform a sensitivity analysis using values of the coefficient that are 0.2 higher and lower than the value from equation 10.A.6. The plus or minus 0.2 range defines the 95% confidence limit for equation 10.A.6 within the domain of the regression data. If the difference between the two ends of this range is substantial, then the conservative value should be used. The expansion coefficient should not be higher than 0.8.

Contraction reach length

Ranges of contraction ratios (CR) for different conditions are presented in Table 10.A.3. These values should be used as starting values. Note that this table does not differentiate on the basis of the degree of constriction. For each range, the higher values are typically associated with higher discharges.

Appendix A Flow Transitions in Bridge Backwater Analysis (continued)

Contraction reach length (continued)

Table 10.A.3
Contraction Ratio

S, ft/mile	n_{ob}/n_{mc}= 1	n_{ob}/n_{mc}= 2	n_{ob}/n_{mc}= 4
1 ft/mile	1.0--2.3	0.8--1.7	0.7--1.3
5 ft/mile	1.0--1.9	0.8--1.5	0.7--1.2
10 ft/mile	1.0--1.9	0.8--1.4	0.7--1.2

When the conditions are within or near those of the data, the contraction reach length can be estimated by

$$L_c = 263 + 38.8\,(F_{c4}/F_{c3}) + 257\,(Q_{ob}/Q)^2 - 58.7\,(n_{ob}/n_c)^{0.5} + 0.161 L_{obs} \qquad \textbf{(10.A.4)}$$

Where:

L_{obs} = average length of obstruction caused by the two bridge approaches, in feet, and
Q_{ob} = the discharge conveyed by the two overbanks, at the approach section (Section 4)
n_{ob} = the Manning's n for the overbanks at Section 4
n_c = the Manning's n for the main channel at Section 4

In cases where the floodplain scale and discharge are significantly larger or smaller than those used in developing the regression formulae, equation 10.A.4 should not be used. The recommended approach is to compute a value from equation 10-A-5 and check it against the values in Table 10.A.3. The contraction ratio should not exceed 2.5:1 nor should it be less than 0.3:1.

$$CR=1.4-0.333(F_{c4}/F_{c3})+1.86(Q_{ob}/Q)^2-0.19(n_{ob}/n_c)^{0.5} \qquad \textbf{(10.A.4)}$$

Contraction Coefficient
The data in the study did not lend itself to regression analysis of the contraction coefficient as for nearly all the data, the value determined was 0.1, which is considered the minimum acceptable value.

Table 10.A.4
Contraction Coefficient Values

Degree of Constriction	Recommended Contraction Coefficient
0.0<b/B<0.25	0.3-0.5
0.25<b/B<0.5	0.1-0.3
0.50 <b/B<1.0	0.1

Appendix B Hydraulics of Bridge Waterways

10.B.1 Introduction

This appendix addresses the manual calculation of bridge backwater as presented in FHWA HDS-1. The information presented in this appendix covers the necessary calculations. The user should refer to the referenced publication for a more complete coverage of the subject.

10.B.2 Hydraulics Of Bridge Waterways

Backwater

The expression for backwater has been formulated by applying the principle of conservation of energy between the point of maximum backwater upstream from the bridge, section 4, and a point downstream from the bridge at which normal stage has been reestablished, section 1 (Figure 10-1a). The expression is reasonably valid if the channel in the vicinity of the bridge is essentially straight, the cross sectional area of the stream is fairly uniform, the gradient of the bottom is approximately constant between sections 1 and 4, the flow is free to contract and expand, there is no appreciable scour of the bed in the constriction and the flow is in the subcritical range.

The expression for computation of backwater upstream from a bridge constricting the flow is as follows:

$$h_4^* = K^* \alpha_3 V_{n3}^2/2g + \alpha_4 [(A_{n3}/A_1)^2 - (A_{n3}/A_4)^2] \; V_{n3}^2/2g$$

h_4^* = total backwater, ft
K^* = total backwater coefficient
α_4 & α_3 = kinetic energy coefficient, as defined below
A_{n3} = gross water area in constriction measured below normal stage, ft^2
V_{n3} = average velocity in constriction* or Q/A_{n2}, ft/s
A_1 = water area at section 4 where normal stage is reestablished, ft^2
A_4 = total water area at section 1, including that produced by the backwater, ft^2

To compute backwater, it is necessary to obtain the approximate value of h_4^* by using the first part of the expression:

$$h_4^* = [K^* \alpha_3 (V_{n3}^2)]/2g$$

The value of A_4 in the second part of expression, which depends on h_4^*, can then be determined and the second term of the expression evaluated:

$$\alpha_4 [(A_{n3}/A_1)^2 - (A_{n3}/A_4)^2] V_{n3}^2/2g$$

This part of the expression represents the difference in kinetic energy between sections 1 and 4, expressed in terms of the velocity head, $V_{n3}^2/2g$.

Appendix B Hydraulics of Bridge Waterways (continued)

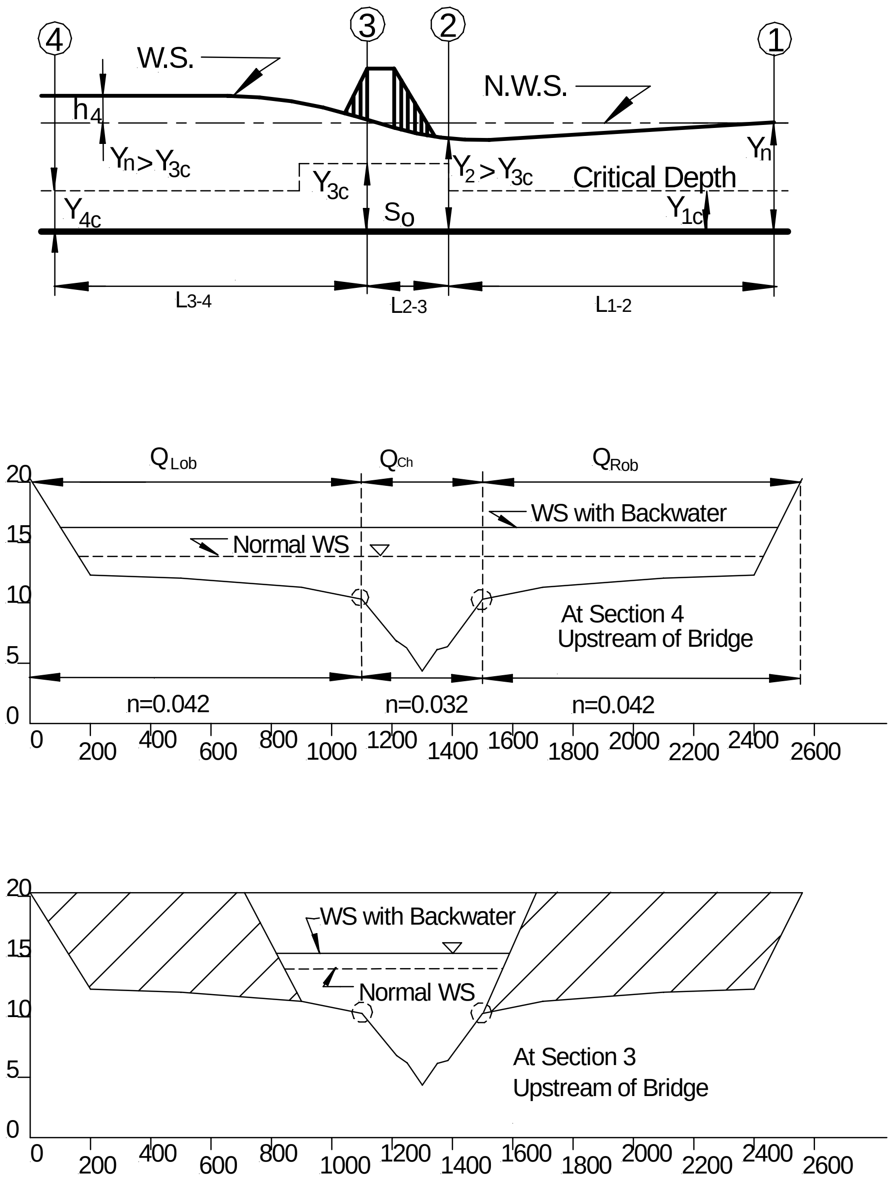


Figure 10-B-1 Normal Crossing: Spillthrough Abutments

Appendix B Hydraulics of Bridge Waterways (continued)

Bridge Opening Ratio

$M = Q_{MC}/(Q_a + Q_{MC} + Q_c)$

Kinetic Energy Coefficient

$\alpha_4 = (qv^2)/QV_4^2$

Where: v = average velocity in a subsection
q = discharge in same subsection
Q = total discharge in river
V_4 = average velocity in river at section 4 or Q/A_4

Width of Constriction

$b = A_{n3}/y$ (Figure 10-B-1)

Backwater Coefficient

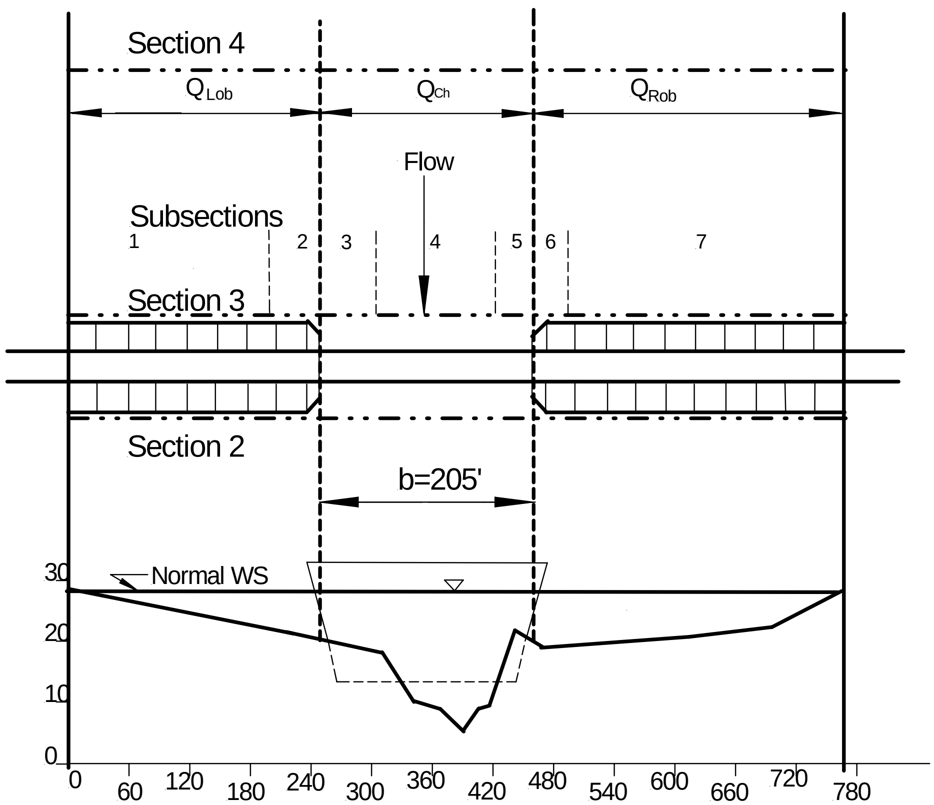
$K^* = K_b + \Delta K_p + \Delta K_s + \Delta K_e$

Where: K_b is the base constriction coefficient
ΔK_p is the pier coefficient
ΔK_s is the skew coefficient
ΔK_e is the eccentricity coefficient
Individual coefficient values are obtained from figures in HDS-1.

Appendix B Hydraulics of Bridge Waterways (continued)

Example 1 The channel crossing is shown in Figure 10-B-2 with the following information: Cross section of river at bridge site (areas, wetted perimeters, and values of Manning's n are given); normal water surface for design = El 28.0 ft at bridge; average slope of river in vicinity of bridge So=2.6 ft/mile or 0.00049 ft/ft; cross section under bridge showing area below normal water surface and width of roadway = 40 ft. The stream is essentially straight, the cross section relatively constant in the vicinity of the bridge, and the crossing is normal to the general direction of flow.

Find the Bridge Backwater caused by this roadway crossing



Subsection	n	a	p
1	0.045	627.4	200.2
2	0.070	285.2	40.1
3	0.070	324.5	40.1
4	0.035	2004.0	145.0
5	0.050	205.8	25.1
6	0.050	539.4	55.1
7	0.045	1677.4	251.0

Figure 10-B-2 Channel Crossing

Appendix B Hydraulics of Bridge Waterways (continued)

Solution

Under the conditions stated, it is permissible to assume that the cross sectional area of the stream at section 1 is the same as that at the bridge. The approach section is then divided into subsections at abrupt changes in depth or channel roughness as shown in Figure 10-B-2. The conveyance of each subsection is computed as shown in columns 1 through 8 of Table 10-B-l. The summation of the individual values in column 8 represents the overall conveyance of the stream at section 1 or K_1 = 879 489. Note that the water interface between subsections is not included in the wetted perimeter. Table 10-B-1 is set up in short form to better demonstrate the method. The actual computation would involve many subsections corresponding to breaks in grade or changes in channel roughness.

Since the slope of the stream is known (0.00049 ft/ft) and the cross sectional area is essentially constant throughout the reach under consideration, it is permissible to solve for the discharge by what is known as the slope-area method or;

$Q = K_1 S_o^{1/2} = 879{,}489 * (0.000\,49)^{1/2} = 19{,}500\ ft^{3/s}$

To compute the kinetic energy coefficient, it is first necessary to complete columns 9, 10, 11 of Table 10-B-1; then:

$$\alpha_4 = \frac{374.895}{19{,}500\ (19{,}500/5{,}664)^2} = 1.62$$

The sum of the individual discharges in column 9 must equal 19,500 ft^3/s. The factor M is the ratio of that portion of the discharge approaching the bridge in width b, to the total discharge of the river:

$M = Q_{MC}/Q \quad M = 12{,}040/19{,}500 = 0.62$

Entering Figure 5 in HDS-1 with $\alpha_4 = 1.61$ and M = 0.62, the value of α_3 is estimated as 1.40.

Entering Figure 6 in HDS-1 with M = 0.62, the base curve coefficient is $K_b = 0.72$ for bridge waterway of 205 ft.

As the bridge is supported by five solid piers, the incremental coefficient (ΔK_p) for this effect is determined. Referring to Figure 10-B-2 and Table 10-B-1: the gross water area under the bridge for normal stage, A_{n3} is 2,534 ft^2 and the area obstructed by the piers, $A_{p,}$ is 180 ft^2; so:

$J = 180/2{,}534 = 0.071$

Entering Figure 7A in HDS-1 with J = 0.071 for solid piers, the reading from the ordinate is $\Delta K = 0.13$. This value is for M = 1.0. Now enter Figure 7B in HDS-1 and obtain the correction factor σ, for M = 0.62 which is 0.84. The incremental backwater coefficient for the five piers, $\Delta K_p = \Delta K\sigma = 0.13 \times 0.84 = 0.11$

Appendix B Hydraulics of Bridge Waterways (continued)

The overall backwater coefficient:

$K^* = K_b + \Delta K_p = 0.72 + 0.11 = 0.83,$

$$V_{n3} = \frac{Q}{A_{n3}} = \frac{19{,}500}{2{,}534} = 7.70 \text{ ft/s}$$

and

$V_{n3}^2/2g = (7.70)^2/2(32.2) = 0.92$ ft

The approximate backwater will be:

$K^*\alpha_3\,(V_{n3}^2/2g) = 0.83 \times 1.40 \times 0.92 = 1.07$ ft.

Substituting values in the second half of expression for difference in kinetic energy between sections 1 and 4 where $A_{n4} = 5664\ ft^2 = A_1$.

$A_4 = 6384\ ft^2$ and $A_{n3} = 2534\ ft^2$

$\alpha_4\,[(A_{n3}/A_1)^2 - (A_{n3}/A_4)^2]\ \ V_{n3}^2/2g$

$1.61\,[(2534/5664)^2 - (2534/6384)^2] \times 0.92$
$= (1.61)(0.042)(0.92)$
$= 0.06$

Then total backwater produced by the bridge is

$h_4^* = 1.07 + 0.06 = 1.13$ ft.

Appendix B Hydraulics of Bridge Waterways (continued)

Table 10-B-1 Calculation Summary

	SUB-SECTION	n	A Ft^2	p Ft	$r = \frac{a}{p_{(ft)}}$ ft	$r^{2/3}$	$k = \frac{1.49 a r^{2/3}}{n}$	$q = Q\frac{k}{k_1}$ cfs	$V = \frac{q}{a}$ fps	qv^2
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Q_c	0-200	0.045	627.4	200.2	3.134	2.142	44,349	983.3	1.57	2,424
	200-240	0.070	285.2	40.1	7.112	3.698	22,359	495.7	1.74	1,501
Q_b	240-280	0.070	324.5	40.1	8.092	4.031	27,732	614.8	1.89	2,196
	280-420	0.035	2004.0	145.0	13.821	5.759	490,492	10,875.2	5.43	320,654
	420-445	0.050	205.8	25.1	8.199	4.066	24,852	551.0	2.68	3,958
Q_a	445-500	0.050	539.4	55.1	9.789	4.576	73,309	1,625.4	3.01	14,726
	500-750	0.045	1677.4	251.0	6.683	3.548	196,396	4,354.6	2.60	29,436
			A_n = 5663.77 ft^2				k_1=879,489	Q=19,500 cfs		Σqv^2= 374,895
			An_3=2534 ft^2					Q_{mc}=12,040 cfs		

$\int_o$ = 0.00049

Appendix C HEC-RAS Examples

Existing - no bridge:

```
            X     X  XXXXXX    XXXX        XXXX       XX      XXXX
            X     X  X        X    X       X   X     X  X    X
            X     X  X        X            X   X    X    X   X
            XXXXXXX  XXXX     X       XXX  XXXX     XXXXXX    XXXX
            X     X  X        X            X  X     X    X        X
            X     X  X        X    X       X   X    X    X        X
            X     X  XXXXXX    XXXX        X    X   X    X   XXXXX


PROJECT DATA
Project Title: HEC18_existing_test
Project File : HEC18EXS_T.prj
Run Date and Time: 3/29/2004 1:41:36 PM

Project in English units

Project Description:
Hydraulics Manual

PLAN DATA

Plan Title: Plan 23
Plan File : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.p23

           Geometry Title: HEC18_Existing_nobridge
           Geometry File : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.g01
           Flow Title    : Flow 01
           Flow File     : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.f01

Plan Summary Information:
Number of:  Cross Sections =    4    Multiple Openings  =    0
            Culverts       =    0    Inline Structures  =    0
            Bridges        =    0    Lateral Structures =    0

Computational Information
    Water surface calculation tolerance  =  0.01
    Critical depth calculation tolerance =  0.01
    Maximum number of iterations         =  20
    Maximum difference tolerance         =  0.3
    Flow tolerance factor                =  0.001

Computation Options
    Critical depth computed only where necessary
    Conveyance Calculation Method: At breaks in n values only
    Friction Slope Method: Average Conveyance
    Computational Flow Regime: Subcritical Flow
```

Appendix C HEC-RAS Examples (continued)

Existing - no bridge:

```
FLOW DATA
Flow Title: Flow 01
Flow File : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.f01
Flow Data (cfs)

  River           Reach           RS                       PF 1
  Test 1          1               21                      30000

Boundary Conditions
  River           Reach           Profile          Upstream        Downstream
  Test 1          1               PF 1
        Normal S = 0.002

GEOMETRY DATA

Geometry Title: HEC18_Existing_nobridge
Geometry File : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.g01

CROSS SECTION
RIVER: Test 1
REACH: 1                  RS: 21
INPUT
Description:
Station Elevation Data    num= 18
Sta     Elev     Sta     Elev     Sta     Elev     Sta    Elev     Sta    Elev
   0    21.7     100    17.7     200    13.7     500   13.45     850   12.79
 900    12.7    1100    11.7    1215     8.2    1250     7.6    1300    5.75
1350    7.55    1385     7.8    1500    11.7    1700    12.7    2100   13.45
2400    13.7    2500    17.7    2600    21.7

Manning's n Values        num=       3
     Sta   n Val      Sta   n Val     Sta   n Val
       0    .042     1100    .032    1500    .042

Bank Sta:         Lengths:                     Coeff
Left   Right     Left   Channel   Right     Contr.   Expan.
1100    1500     650     650        650        .1       .3

  Profile #PF 1
            Pos          Left Sta    Right Sta       Flow       Area
W.P.         Percent       Hydr   Velocity
             Conv         Depth
                          (ft)        (ft)          (cfs)     (sq ft)
(ft)                      (ft)       (ft/s)
  1           LOB          0.00        220.00        99.01       62.70
 59.42       0.33       1.06       1.58

  2           LOB         220.00       440.00       798.73      370.44
220.00       2.66       1.68       2.16

  3           LOB         440.00       660.00      1001.30      424.25
220.00       3.34       1.93       2.36

  4           LOB         660.00       880.00      1376.84      513.58
220.00       4.59       2.33       2.68
```

Appendix C HEC-RAS Examples (continued)

Existing - no bridge:

```
  Profile #PF 1
            Pos          Left Sta    Right Sta       Flow       Area
W.P.         Percent        Hydr   Velocity
             Conv          Depth
                           (ft)        (ft)          (cfs)    (sq ft)
(ft)                       (ft)       (ft/s)

  5           LOB          880.00      1100.00      2124.50     666.25
220.00       7.08        3.03        3.19

  6           Chan         1100.00     1200.00      3057.04     509.72
100.05      10.19        5.10        6.00

  7           Chan         1200.00     1300.00      6357.06     790.88
100.05      21.19        7.91        8.04

  8           Chan         1300.00     1400.00      6555.86     805.61
100.04      21.85        8.06        8.14

  9           Chan         1400.00     1500.00      3232.60     527.12
100.06      10.78        5.27        6.13

  10          ROB          1500.00     1720.00      2124.41     666.24
220.00       7.08        3.03        3.19

  11          ROB          1720.00     1940.00      1374.16     512.99
220.00       4.58        2.33        2.68

  12          ROB          1940.00     2160.00      1000.75     424.11
220.00       3.34        1.93        2.36

  13          ROB          2160.00     2380.00       798.73     370.44
220.00       2.66        1.68        2.16

  14          ROB          2380.00     2600.00        99.01      62.70
 59.42       0.33        1.06        1.58


Warning: The energy loss was greater than 1.0 ft (0.3 m). between the current
and previous cross section.
         This may indicate the need for additional cross sections.

CROSS SECTION
RIVER: Test 1
REACH: 1                   RS: 14.5
INPUT
Description: Downstream Section
Station Elevation Data    num = 18
  Sta    Elev      Sta    Elev      Sta    Elev      Sta    Elev      Sta    Elev
    0    20.4      100    16.4      200    12.4      500   12.15      850   11.49
  900    11.4     1100    10.4     1215     6.9     1250     6.3     1300    4.45
 1350    6.25     1385     6.5     1500    10.4     1700    11.4     2100   12.15
 2400    12.4     2500    16.4     2600    20.4
```

Appendix C HEC-RAS Examples (continued)

Existing - no bridge

Manning's n Values num= 3

Sta	n Val	Sta	n Val	Sta	n Val
0	.042	1100	.032	1500	.042

Bank Sta: Left	Right	Lengths: Left	Channel	Right	Coeff Contr.	Expan.
1100	1500	100	100	100	.1	.3

Profile #PF 1

	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth (ft)	Velocity (ft/s)
1	LOB	0.00	220.00	99.03	62.71	59.42	0.33	1.06	1.58
2	LOB	220.00	440.00	798.81	370.48	220.00	2.66	1.68	2.16
3	LOB	440.00	660.00	1001.37	424.28	220.00	3.34	1.93	2.36
4	LOB	660.00	880.00	1376.90	513.62	220.00	4.59	2.33	2.68
5	LOB	880.00	1100.00	2124.55	666.29	220.00	7.08	3.03	3.19
6	Chan	1100.00	1200.00	3056.99	509.74	100.05	10.19	5.10	6.00
7	Chan	1200.00	1300.00	6356.84	790.89	100.05	21.19	7.91	8.04
8	Chan	1300.00	1400.00	6555.63	805.63	100.04	21.85	8.06	8.14
9	Chan	1400.00	1500.00	3232.55	527.13	100.06	10.78	5.27	6.13
10	ROB	1500.00	1720.00	2124.46	666.27	220.00	7.08	3.03	3.19
11	ROB	1720.00	1940.00	1374.23	513.02	220.00	4.58	2.33	2.68
12	ROB	1940.00	2160.00	1000.83	424.15	220.00	3.34	1.93	2.36
13	ROB	2160.00	2380.00	798.80	370.48	220.00	2.66	1.68	2.16
14	ROB	2380.00	2600.00	99.03	62.71	59.42	0.33	1.06	1.58

Appendix C HEC-RAS Examples (continued)

Existing - no bridge:

```
CROSS SECTION
RIVER: Test 1
REACH: 1                  RS: 13.5
INPUT
Description: Downstream Section
Station Elevation Data    num = 18
  Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
    0    20.2     100    16.2     200    12.2     500   11.95     850   11.29
  900    11.2    1100    10.2    1215     6.7    1250     6.1    1300    4.25
 1350    6.05    1385     6.3    1500    10.2    1700    11.2    2100   11.95
 2400    12.2    2500    16.2    2600    20.2

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042

Bank Sta:         Lengths:                     Coeff
Left   Right    Left   Channel   Right     Contr.   Expan.
1100    1500     600     600     600         .1       .3

Warning: The energy loss was greater than 1.0 ft (0.3 m). between the current
and previous cross section.
         This may indicate the need for additional cross sections.
```

```
CROSS SECTION
RIVER: Test 1
REACH: 1                  RS: 7.5
INPUT
Description: Downstream Section
Station Elevation Data    num = 18
  Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
    0      19     100      15     200      11     500   10.75     850   10.09
  900      10    1100       9    1215     5.5    1250     4.9    1300    3.05
 1350    4.85    1385     5.1    1500       9    1700      10    2100   10.75
 2400      11    2500      15    2600      19

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042

Bank Sta:         Lengths:                     Coeff
Left   Right    Left   Channel   Right     Contr.   Expan.
1100    1500       0       0       0         .1       .3
```

Appendix C HEC-RAS Examples (continued)

Existing - no bridge:

SUMMARY OF MANNING'S N VALUES
River: Test 1

Reach	River Sta.	n1	n2	n3
1	21	.042	.032	.042
1	14.5	.042	.032	.042
1	13.5	.042	.032	.042
1	7.5	.042	.032	.042

SUMMARY OF REACH LENGTHS
River: Test 1

Reach	River Sta.	Left	Channel	Right
1	21	650	650	650
1	14.5	100	100	100
1	13.5	600	600	600
1	7.5	0	0	0

SUMMARY OF CONTRACTION AND EXPANSION COEFFICIENTS
River: Test 1

Reach	River Sta.	Contr.	Expan.
1	21	.1	.3
1	14.5	.1	.3
1	13.5	.1	.3
1	7.5	.1	.3

Profile Output Table - Q+Flow Dist.

Reach	River Sta	Profile	E.G. Elev (ft)	E.G. Slope (ft/ft)	W.S. Elev (ft)	Crit W.S. (ft)	Min Ch El (ft)
1	21	PF 1	15.84	0.002000	15.28		5.75
1	14.5	PF 1	14.54	0.002000	13.98		4.45
1	13.5	PF 1	14.34	0.002000	13.78		4.25
1	7.5	PF 1	13.14	0.002004	12.57	11.85	3.05

Reach	River Sta	Q Left (cfs)	Vel Left (ft/s)	Q Channel (cfs)	Vel Chnl (ft/s)	Q Right (cfs)	Vel Right (ft/s)	Top Width (ft)
1	21	5400.38	2.65	19202.56	7.29	5397.06	2.65	2278.78
1	14.5	5400.65	2.65	19202.01	7.29	5397.34	2.65	2278.78
1	13.5	5400.55	2.65	19202.21	7.29	5397.24	2.65	2278.78
1	7.5	5396.70	2.65	19209.92	7.30	5393.38	2.65	2278.67

Reach	River Sta	Area Left (sq ft)	Area Channel (sq ft)	Area Right (sq ft)
1	21	2037.23	2633.33	2036.48
1	14.5	2037.38	2633.39	2036.63
1	13.5	2037.33	2633.37	2036.58
1	7.5	2035.22	2632.47	2034.47

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
 X     X  XXXXXX    XXXX        XXXX       XX      XXXX
 X     X  X        X    X       X   X     X  X    X
 X     X  X        X            X   X    X    X   X
 XXXXXXX  XXXX     X       XXX  XXXX     XXXXXX    XXXX
 X     X  X        X            X  X     X    X        X
 X     X  X        X    X       X   X    X    X        X
 X     X  XXXXXX    XXXX        X    X   X    X   XXXXX
```

```
PROJECT DATA
Project Title: HEC18_existing_test
Project File : HEC18EXS_T.prj
Run Date and Time: 2/26/2004 8:58:57 AM

Project in English units

Project Description:
Hydraulics Manual

PLAN DATA

Plan Title: Plan 11
Plan File : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.p11

           Geometry Title: HEC18_Br1_addsect
           Geometry File : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.g08

           Flow Title    : Flow 01
           Flow File     : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.f01

Plan Summary Information:
Number of:  Cross Sections =    9    Multiple Openings  =    0
            Culverts       =    0    Inline Structures  =    0
            Bridges        =    1    Lateral Structures =    0

Computational Information
    Water surface calculation tolerance  =  0.01
    Critical depth calculation tolerance =  0.01
    Maximum number of iterations         =  20
    Maximum difference tolerance         =  0.3
    Flow tolerance factor                =  0.001

Computation Options
    Critical depth computed only where necessary
    Conveyance Calculation Method: At breaks in n values only
    Friction Slope Method:         Average Conveyance
    Computational Flow Regime:     Subcritical Flow
```

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
FLOW DATA

Flow Title: Flow 01
Flow File : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.f01

Flow Data (cfs)
  River           Reach           RS                      PF 1
  Test 1          1               21                     30000


Boundary Conditions
  River           Reach           Profile          Upstream         Downstream
  Test 1          1               PF 1
        Known WS = 12.57


GEOMETRY DATA
Geometry Title: HEC18_Br1_addsect
Geometry File : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.g08

CROSS SECTION
RIVER: Test 1
REACH: 1                  RS: 21

INPUT
Description:
Station Elevation Data    num = 17
  Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
    0    21.7     100    17.7     200    13.7     500   13.45     900    12.7
 1100    11.7    1215     8.2    1250     7.6    1300    5.75    1350    7.55
 1385     7.8    1500    11.7    1700    12.7    2100   13.45    2400    13.7
 2500    17.7    2600    21.7

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042

 Bank Sta:
 Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
 1100    1500              200     200     200              .1       .3
```

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
  Profile #PF 1
                Pos           Left Sta    Right Sta        Flow        Area
W.P.    Percent Conv.   Hydr Depth Velocity
                           (ft)        (ft)            (cfs)     (sq ft)
(ft)                    (ft)        (ft/s)
  1             LOB        0.00        220.00         184.17      128.39
82.58       0.61        1.56        1.43

  2             LOB        220.00      440.00        1163.05      574.10
220.00       3.88       2.61        2.03

  3             LOB        440.00      660.00        1349.84      627.77
220.00       4.50       2.85        2.15

  4             LOB        660.00      880.00        1683.15      716.64
220.00       5.61       3.26        2.35

  5             LOB        880.00      1100.00       2324.83      869.89
220.00       7.75       3.95        2.67

  6             Chan       1100.00     1200.00       2817.26      602.29
100.05       9.39       6.02        4.68

  7             Chan       1200.00     1300.00       5334.74      883.45
100.05      17.78       8.83        6.04

  8             Chan       1300.00     1400.00       5484.01      898.18
100.04      18.28       8.98        6.11

  9             Chan       1400.00     1500.00       2953.93      619.69
100.06       9.85       6.20        4.77

  10            ROB        1500.00     1720.00       2324.83      869.89
220.00       7.75       3.95        2.67

  11            ROB        1720.00     1940.00       1683.15      716.64
220.00       5.61       3.26        2.35

  12            ROB        1940.00     2160.00       1349.84      627.77
220.00       4.50       2.85        2.15

  13            ROB        2160.00     2380.00       1163.05      574.10
220.00       3.88       2.61        2.03

  14            ROB        2380.00     2600.00        184.17      128.39
82.58       0.61       1.56        1.43

CROSS SECTION
RIVER: Test 1
REACH: 1                  RS: 19

INPUT
Description: Downstream Section
Station Elevation Data    num = 17
 Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
   0    21.3     100    17.3     200    13.3     500   13.05     900    12.3
1100    11.3    1215     7.8    1250     7.2    1300    5.35    1350    7.15
1385     7.4    1500    11.3    1700    12.3    2100   13.05    2400    13.3
```

```
2500    17.3    2600    21.3
```

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042
Bank Sta:
Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
1100    1500              200     200     200              .1       .3

Ineffective Flow     num=       2
   Sta L   Sta R    Elev  Permanent
       0     406    22.9        T
    2196    2600    22.9        T

CROSS SECTION
RIVER: Test 1
REACH: 1                  RS: 17
INPUT
Description: Downstream Section
Station Elevation Data    num = 17
 Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
   0    20.9     100    16.9     200    12.9     500   12.65     900    11.9
1100    10.9    1215     7.4    1250     6.8    1300    4.95    1350    6.75
1385       7    1500    10.9    1700    11.9    2100   12.65    2400    12.9
2500    16.9    2600    20.9

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042
Bank Sta:
Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
1100    1500              200     200     200              .1       .3

Ineffective Flow     num=       2
   Sta L   Sta R    Elev  Permanent
       0     606    22.5        T
    1796    2600    22.5        T

CROSS SECTION
RIVER: Test 1
REACH: 1                  RS: 15.0
INPUT
Description: Downstream Section
Station Elevation Data    num = 17
 Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
   0    20.5     100    16.5     200    12.5     500   12.25     900    11.5
1100    10.5    1215       7    1250     6.4    1300    4.55    1350    6.35
1385     6.6    1500    10.5    1700    11.5    2100   12.25    2400    12.5
2500    16.5    2600    20.5

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042
Bank Sta:
Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
1100    1500               50      50      50              .1       .3

Ineffective Flow     num = 2
   Sta L   Sta R    Elev  Permanent
       0     806    22.1        T
```

```
    1596    2600    22.1       T
```

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
CROSS SECTION
RIVER: Test 1
REACH: 1                    RS: 14.5

INPUT
Description: Downstream Section
Station Elevation Data    num = 17
 Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
   0    20.4     100    16.4     200    12.4     500   12.15     900    11.4
1100    10.4    1215     6.9    1250     6.3    1300    4.45    1350    6.25
1385     6.5    1500    10.4    1700    11.4    2100   12.15    2400    12.4
2500    16.4    2600    20.4

REACH: 1                    RS: 14.5

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042
Bank Sta:
Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
1100    1500              100     100     100              .1       .3

Ineffective Flow     num=       2
   Sta L   Sta R    Elev  Permanent
       0     856      22       T
    1546    2600      22       T

  Profile #PF 1
             Pos          Left Sta    Right Sta        Flow        Area
W.P.    Percent Conv.   Hydr Depth Velocity
                          (ft)        (ft)            (cfs)     (sq ft)
(ft)                      (ft)       (ft/s)
  1           LOB         0.00       214.00          0.00       71.08
 61.24       0.00       1.16       0.00

  2           LOB       214.00       428.00          0.00      425.65
214.00       0.00       1.99       0.00

  3           LOB       428.00       642.00          0.00      474.31
214.00       0.00       2.22       0.00

  4           LOB       642.00       856.00          0.00      557.48
214.00       0.00       2.61       0.00

  5           LOB       856.00      1100.00       3480.75      802.90
244.00      11.60       3.29       4.34

  6          Chan      1100.00      1200.00       4190.29      540.99
100.05      13.97       5.41       7.75

  7          Chan      1200.00      1300.00       8417.30      822.14
100.05      28.06       8.22      10.24

  8          Chan      1300.00      1400.00       8670.47      836.88
100.04      28.90       8.37      10.36

  9          Chan      1400.00      1500.00       4416.87      558.38
100.06      14.72       5.58       7.91
```

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
  Profile #PF 1
             Pos            Left Sta    Right Sta        Flow        Area
W.P.    Percent Conv.   Hydr Depth Velocity
                          (ft)        (ft)           (cfs)    (sq ft)
(ft)                      (ft)       (ft/s)
  10          ROB         1500.00     1546.00       824.33     173.57
 46.00       2.75       3.77       4.75

  11          ROB         1546.00     1756.80         0.00     665.09
210.80       0.00       3.16       0.00

  12          ROB         1756.80     1967.60         0.00     544.72
210.80       0.00       2.58       0.00

  13          ROB         1967.60     2178.40         0.00     464.60
210.80       0.00       2.20       0.00

  14          ROB         2178.40     2389.20         0.00     418.44
210.80       0.00       1.99       0.00

  15          ROB         2389.20     2600.00         0.00      65.01
58.04       0.00       1.12       0.00


BRIDGE

RIVER: Test 1
REACH: 1                  RS: 14.0

INPUT
Description: Bridge#1
B
Distance from Upstream XS = 30
Deck/Roadway Width        = 40
Weir Coefficient          = 2.6
Upstream Deck/Roadway Coordinates
    Num = 8
     Sta Hi Cord Lo Cord     Sta Hi Cord Lo Cord     Sta Hi Cord Lo Cord
       0      22      10     100      22      10     500      22      10
     850      22      10   850.1      22      18    1500      22      18
  1500.1      22      10    2600      22      10

Upstream Bridge Cross-Section Data
Station Elevation Data    num = 17
 Sta    Elev      Sta    Elev      Sta    Elev      Sta    Elev      Sta    Elev
   0    20.4      100    16.4      200    12.4      500   12.15      900    11.4
1100    10.4     1215     6.9     1250     6.3     1300    4.45     1350    6.25
1385     6.5     1500    10.4     1700    11.4     2100   12.15     2400    12.4
2500    16.4     2600    20.4

Manning's n Values        num = 3
     Sta   n Val      Sta   n Val      Sta   n Val
       0    .042     1100    .032     1500    .042
```

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
Bank Sta: Left   Right    Coeff Contr.   Expan.
          1100    1500             .1       .3

Ineffective Flow     num = 2
   Sta L   Sta R    Elev  Permanent
       0     856      22        T
    1546    2600      22        T

Downstream Deck/Roadway Coordinates
    Num = 8
     Sta Hi Cord Lo Cord     Sta Hi Cord Lo Cord     Sta Hi Cord Lo Cord
       0      22      10     100      22      10     500      22      10
     850      22      10   850.1      22      18    1500      22      18
  1500.1      22      10    2600      22      10

Downstream Bridge Cross-Section Data
Station Elevation Data    num = 17
 Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
   0    20.2     100    16.2     200    12.2     500   11.95     900    11.2
1100    10.2    1215     6.7    1250     6.1    1300    4.25    1350    6.05
1385     6.3    1500    10.2    1700    11.2    2100   11.95    2400    12.2
2500    16.2    2600    20.2

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042

Bank Sta: Left   Right    Coeff Contr.   Expan.
          1100    1500             .1       .3

Ineffective Flow     num = 2
   Sta L   Sta R    Elev  Permanent
       0     876      22        T
    1526    2600      22        T

Upstream Embankment side slope              = 2 horiz. to 1.0 vertical
Downstream Embankment side slope            = 2 horiz. to 1.0 vertical
Maximum allowable submergence for weir flow =     .95
Elevation at which weir flow begins         =
Energy head used in spillway design         =
Spillway height used in design              =
Weir crest shape                            = Broad Crested

Number of Abutments = 1

Abutment Data
Upstream     num = 4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
     816      18     817       4    1515       4    1516      18


Downstream     num = 4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
     816      18     817       4    1515       4    1516      18

Number of Bridge Coefficient Sets = 1
```

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
Low Flow Methods and Data
       Energy
Selected Low Flow Methods = Highest Energy Answer

High Flow Method
       Energy Only

Additional Bridge Parameters
       Add Friction component to Momentum
       Do not add Weight component to Momentum
       Class B flow critical depth computations use critical depth
       inside the bridge at the upstream end
       Criteria to check for pressure flow = Upstream energy grade line

CROSS SECTION
RIVER: Test 1
REACH: 1                 RS: 13.5
INPUT
Description: Downstream Section
Station Elevation Data    num = 17
 Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
   0    20.2     100    16.2     200    12.2     500   11.95     900    11.2
1100    10.2    1215     6.7    1250     6.1    1300    4.25    1350    6.05
1385     6.3    1500    10.2    1700    11.2    2100   11.95    2400    12.2
2500    16.2    2600    20.2

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042
Bank Sta:
Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
1100    1500             200     200     200              .1       .3

Ineffective Flow     num = 2
   Sta L   Sta R    Elev  Permanent
       0     876      22       T
    1526    2600      22       T


Warning: The velocity head has changed by more than 0.5 ft (0.15 m).  This may
indicate the need for additional cross sections.

Warning: The conveyance ratio (upstream conveyance divided by downstream
conveyance) is less than 0.7 or greater than 1.4.  This may indicate the need
for additional cross sections.

Warning: The energy loss was greater than 1.0 ft (0.3 m). between the current
and previous cross section.  This may indicate the need for additional cross
sections.
```

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
CROSS SECTION

RIVER: Test 1
REACH: 1                    RS: 11.5

INPUT
Description:
Station Elevation Data    num = 17
 Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
   0    19.8     100    15.8     200    11.8     500   11.55     900    10.8
1100     9.8    1215     6.3    1250     5.7    1300    3.85    1350    5.65
1385     5.9    1500     9.8    1700    10.8    2100   11.55    2400    11.8
2500    15.8    2600    19.8

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042
Bank Sta:
Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
1100    1500             200     200     200               .1       .3

CROSS SECTION
RIVER: Test 1
REACH: 1                    RS: 9.5

INPUT
Description:
Station Elevation Data    num = 17
 Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
   0    19.4     100    15.4     200    11.4     500   11.15     900    10.4
1100     9.4    1215     5.9    1250     5.3    1300    3.45    1350    5.25
1385     5.5    1500     9.4    1700    10.4    2100   11.15    2400    11.4
2500    15.4    2600    19.4

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042
Bank Sta:
Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
1100    1500             200     200     200               .1       .3


CROSS SECTION
RIVER: Test 1
REACH: 1                    RS: 7.5

INPUT
Description: Downstream Section
Station Elevation Data    num = 17
 Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
   0      19     100      15     200      11     500   10.75     900      10
1100       9    1215     5.5    1250     4.9    1300    3.05    1350    4.85
1385     5.1    1500       9    1700      10    2100   10.75    2400      11
2500      15    2600      19

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042
```

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
Bank Sta:
Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
1100    1500                 0       0       0              .1       .3
```

```
Warning: The parabolic search method failed to converge on critical depth.
The program will try the cross section slice/secant method to find critical
depth.
```

SUMMARY OF MANNING'S N VALUES

River: Test 1

Reach	River Sta.	n1	n2	n3
1	21	.042	.032	.042
1	19	.042	.032	.042
1	17	.042	.032	.042
1	15.0	.042	.032	.042
1	14.5	.042	.032	.042
1	14.0	Bridge		
1	13.5	.042	.032	.042
1	11.5	.042	.032	.042
1	9.5	.042	.032	.042
1	7.5	.042	.032	.042

SUMMARY OF REACH LENGTHS

River: Test 1

Reach	River Sta.	Left	Channel	Right
1	21	200	200	200
1	19	200	200	200
1	17	200	200	200
1	15.0	50	50	50
1	14.5	100	100	100
1	14.0	Bridge		
1	13.5	200	200	200
1	11.5	200	200	200
1	9.5	200	200	200
1	7.5	0	0	0

SUMMARY OF CONTRACTION AND EXPANSION COEFFICIENTS

River: Test 1

Reach	River Sta.	Contr.	Expan.
1	21	.1	.3
1	19	.1	.3
1	17	.1	.3
1	15.0	.1	.3
1	14.5	.1	.3
1	14.0	Bridge	
1	13.5	.1	.3
1	11.5	.1	.3
1	9.5	.1	.3
1	7.5	.1	.3

Appendix C HEC-RAS Examples (continued)

With Bridge:

Profile Output Table - Q+Flow Dist.

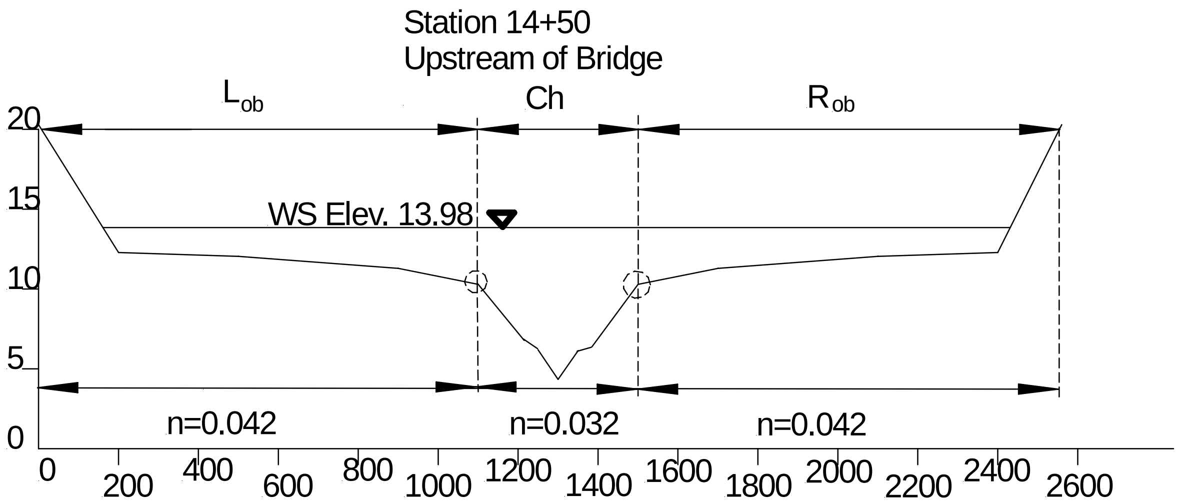
Reach	River Sta	Profile	E.G. Elev (ft)	E.G. Slope (ft/ft)	W.S. Elev (ft)	Crit W.S. (ft)	Min Ch El (ft)	Q Left (cfs)	Vel Left (ft/s)	Q Channel (cfs)	Vel Chnl (ft/s)	Q Right (cfs)	Vel Right (ft/s)	Top Width (ft)	Area Left (sq ft)	Area Channel (sq ft)	Area Right (sq ft)
1	21	PF 1	16.50	0.000963	16.20		5.75	6705.03	2.30	16589.93	5.52	6705.04	2.30	2325.06	2916.79	3003.61	2916.79
1	19	PF 1	16.30	0.001005	15.96		5.35	6220.25	2.58	17547.16	5.72	6232.59	2.57	2332.94	3068.77	3066.64	3068.77
1	17	PF 1	16.04	0.001401	15.51		4.95	5664.31	3.14	20512.76	6.73	3822.93	3.29	2330.65	3024.45	3048.28	3024.45
1	15.0	PF 1	15.65	0.002344	14.71		4.55	4111.40	3.97	24249.83	8.40	1638.77	4.30	2310.66	2640.51	2888.36	2640.51
1	14.5	PF 1	15.49	0.003068	14.29	13.00	4.45	3480.75	4.34	25694.93	9.32	824.33	4.75	2294.41	2331.42	2758.40	2331.42
1	14.0		Bridge														
1	13.5	PF 1	14.98	0.005813	13.08		4.25	2457.32	4.73	27149.93	11.54	392.75	5.37	2243.82	1385.89	2353.70	1385.89
1	11.5	PF 1	13.94	0.002003	13.37		3.85	5394.79	2.65	19210.43	7.30	5394.78	2.65	2278.69	2034.91	2632.66	2034.91
1	9.5	PF 1	13.54	0.002007	12.97		3.45	5391.64	2.65	19216.73	7.30	5391.64	2.65	2278.60	2033.19	2631.93	2033.19
1	7.5	PF 1	13.14	0.002010	12.57	11.85	3.05	5388.18	2.65	19223.64	7.31	5388.18	2.65	2278.50	2031.31	2631.13	2031.31

Appendix D Abutment Scour

HEC-18 example:

Determine the abutment scour for the given conditions:

The approach channel cross-section, station 14+50, without the bridge is shown below:



Stream Slope: S = 0.002'/'

Table 1
Cross-section Description

Point	Station	Elevation	Manning's n
1	0	20.40	0.042
2	200	12.40	0.042
3	500	12.15	0.042
4	900	11.4	0.042
5	1100	10.40	0.032
6	1215	6.90	0.032
7	1250	6.30	0.032
8	1300	4.45	0.032
9	1350	6.25	0.032
10	1385	6.50	0.032
11	1500	10.40	0.042
12	1700	11.40	0.042
13	2100	12.15	0.042
14	2400	12.40	0.042
15	2600	20.40	0.042

Appendix D Abutment Scour (continued)

Discharge: QT = 30,000 cfs.
Unencroached condition:
Flow distribution:

From HEC-RAS for existing conditions at Section 14+50, the water surface elevation is 13.98.

Table 2
HEC-RAS
Without bridge

Slice ID.	Left Station	Right Station	Flow	Area	Hyd. Depth	Velocity
1	0	220	99.1	62.74	1.06	1.58
2	220	440	799.1	370.6	1.68	2.16
3	440	660	1001.2	424.3	1.93	2.36
4	660	880	1374.6	513.1	2.33	2.68
5	880	1100	2121.8	666.4	3.03	3.19
6	1100	1200	3157.2	507.8	5.10	6.00
7	1200	1300	3232.7	527.2	7.91	8.08
8	1300	1400	6555.6	805.7	8.06	8.14
9	1400	1500	3232.7	527.2	5.27	6.13
10	1500	1720	2124.8	666.4	3.03	3.19
11	1720	1940	1374.6	513.1	2.33	2.68
12	1940	2160	1001.2	424.3	1.93	2.36
13	2160	2380	799.1	370.6	1.68	2.16
14	2380	2600	99.1	62.7	1.06	1.58

Q_l= 5395.8 cfs; Q_{mc}= 19,205.4 cfs; Q_r = 5398.8 cfs

650’ BRIDGE

The bridge is 650 long between the faces of abutments. The left abutment is set 200 feet left of bank, station 850. Right abutment is at the bankline, station 1500. The bridge has 6 stem wall piers that are 5 feet thick and 40 feet long. Set the piers at the following stations: 942.5, 1035.5, 1128.5, 1221.5, 1314.5, and 1407.5.

For abutment scour prediction for foundation design, assume embankment in front of abutment is scoured; therefore the abutment scour is calculated for a vertical face. Elevation at 820 = 11.55’

Encroached Section:

At the left abutment: the obstructed flow, Station 0 to Station 850.
Q_{obl} = 5395.8-2121.8-(30/220)*(1374.6) = 3087 cfs.
The obstructed area = 62.7 + 370.6 + 424.3 + 513. 1+ (190/220)*666.4 = 1946.2 sq.ft.

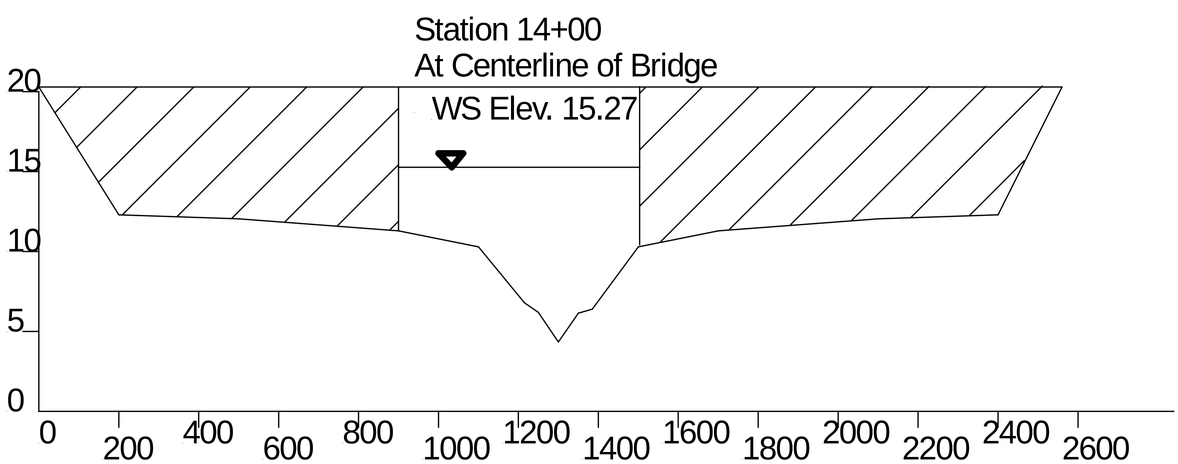
At the right abutment the obstructed flow, station 1500 to station 2600 is Q_{obr} = 5399 cfs.
The obstructed area = 666.4 + 513.1 + 424.3 + 370.6 + 62.7 = 2037.1 sq.ft.

Appendix D Abutment Scour (continued)

The distance from the bridge to the upstream section is 30 feet. Use a 1:1 contraction ratio, therefore at River Station 14.50, set the ineffective flow at 820 and 1530. At section 21.0 the encroachment stations are 170 and 2180. For the downstream section, use an expansion coefficient of 2:1, this is from the expansion coefficient table for b/B=0.25, slope = 10 feet/mile and N_{ob}/N_{mc}=1.3. Range of expansion coefficients is 1.3 to 2.0.

Table 3
Encroachment Stations
650' Bridge

River Station	Left Encroachment Station	Right Encroachment Station
7+50	535	1815
9+50	635	1715
11+50	735	1615
13+50	835	1515
13+80, BD	850	1500
14+20, BU	850	1500
14+50	820	1530
15+00	770	1580
17+00	570	1780
19+00	370	1980
21+00	170	2180



(water surface at 14.83')

Appendix D Abutment Scour (continued)

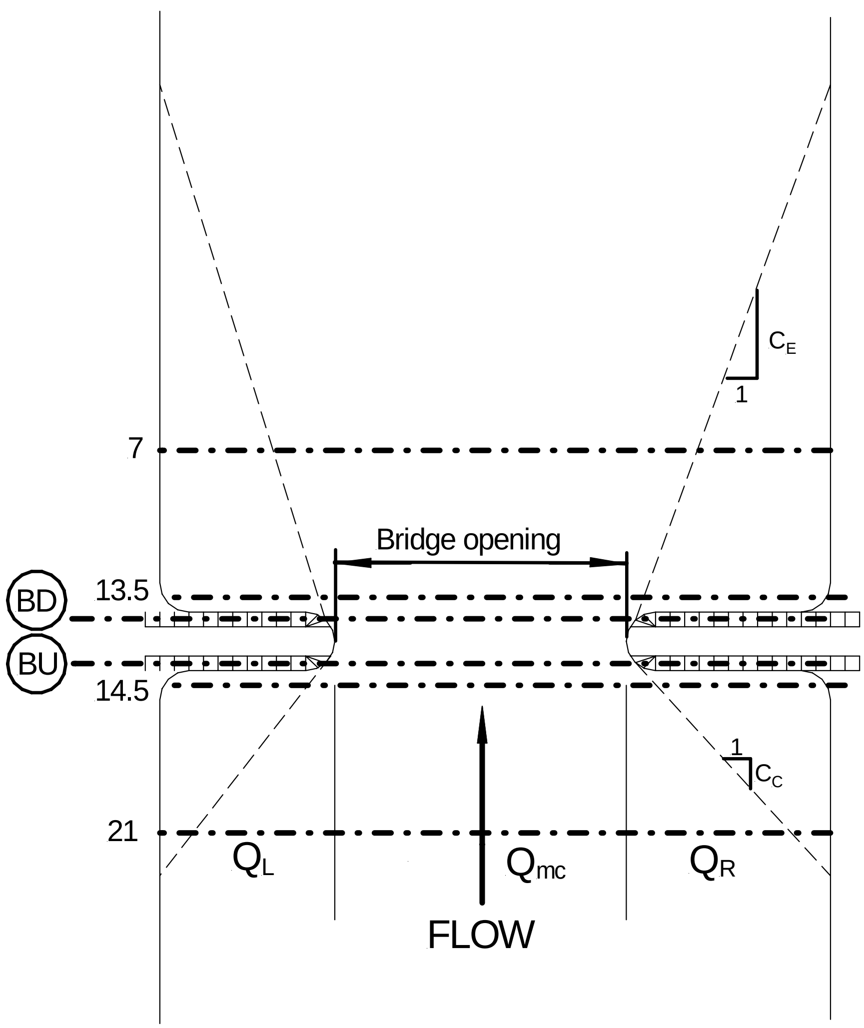
Running HEC-RAS for the encroachments defined above results in the following conditions through the bridge opening: Station 14.50

Table 4
HEC-RAS
650' Bridge

Water Surface elevation 14.83

Left Station	Right Station	Discharge	Hydraulic Depth	Velocity
820	1100	4305	3.76	4.09
1100	1200	4245	5.95	7.14
1200	1300	8091	8.76	9.24
1300	1400	8319	8.91	9.34
1400	1500	4453	6.12	7.27
1500	1530	588	4.35	4.50

The unit discharge at the right abutment: q = (4453+588)/130 = 38.6 cfs/ft.



Appendix D Abutment Scour (continued)

Section 21.   WSE = 16.37

Pos - Flow	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P (ft)	Convey. Per cent	Hydr Depth (ft)	Vel. (ft/s)
1 LOB	0	170.0	0	26.8	536.7	0	0.73	0
2 LOB	170.0	402.5	1260.2	618.8	232.5	4.20	2.66	2.04
3 LOB	402.5	635.0	1514.6	691.0	232.5	5.05	2.97	2.19
4 LOB	635.0	867.5	1883.0	787.4	232.5	6.28	3.39	2.39
5 LOB	867.5	1100.0	2580.4	951.2	232.5	8.60	4.09	2.71
6 CHAN	1100.0	1200.0	2890.5	618.7	100.0	9.63	6.19	4.67
7 CHAN	1200.0	1300.0	5396.5	899.9	100.0	17.99	9.00	6.00
8 CHAN	1300.0	1400.0	5544.7	914.6	100.0	18.48	9.15	6.06
9 CHAN	1400.0	1500.0	3026.9	636.1	100.1	10.09	6.36	4.76
10 ROB	1500.0	1726.6	2541.4	930.2	226.7	8.47	4.10	2.73
11 ROB	1726.6	1953.3	1860.2	771.4	226.7	6.20	3.40	2.41
12 ROB	1953.3	2180.0	1501.6	678.4	226.7	5.01	2.99	2.21
13 ROB	2180.0	2390.0	0	579.9	210.0	0	2.76	0
14 ROB	2390.0	2600.0	0	115.5	76.7	0	1.51	0

Profile Output Table - Q+Flow Dist.

```
Reach      River Sta    Profile      E.G. Elev  E.G. Slope  W.S.E Crit  W.S.
                                       (ft)        (ft/ft)      (ft)    (ft)
Min Ch El    Q Left      Vel Left     Q Channel   Vel Chnl   Q Right    Vel Right
 (ft)        (cfs)       (ft/s)      (cfs)       (ft/s)      (cfs)      (ft/s)
Top Width   Area Left   Area Channel  Area Right
 (ft)        (sq ft)     (sq ft)       (sq ft)
-----------------------------------------------------------------------------------
1             21           PF 1         16.67      0.000925   16.37      14.52
5.75          7238.21      2.37       16858.61      5.49     5903.18      2.48
2333.28     3075.32        3069.35      3075.32

1             19           PF 1         16.47      0.000988    16.12
5.35          6889.77      2.62       18007.81      5.75     5102.42      2.74
2341.00     3224.96        3131.15      3224.96

1             17           PF 1         16.24      0.001195    15.78
4.95          6189.55      3.02       20048.55      6.36     3761.89      3.19
2343.79     3279.10        3153.45      3279.10

1             15.0         PF 1         15.92      0.001858    15.13
4.55          4832.31      3.77       23709.76      7.76     1457.94      4.11
2331.52     3041.27        3055.25      3041.27

1             14.5         PF 1         15.80      0.002280    14.83      13.05
4.45          4305.01      4.09       25107.16      8.44      587.83      4.50
2321.34     2845.32        2973.88      2845.32

1             14.0                                  Bridge

1             13.5          PF 1          15.31    0.003157             14.08
4.25          3733.31       4.35      25986.15      9.44      280.54      4.87
2293.78     2319.55        2753.38      2319.55

1             11.5          PF 1          14.66    0.002665    13.68      12.45
3.85          4348.98       3.87      23883.56      8.67     1767.46      4.28
2293.85     2320.90        2753.95      2320.90
```

Appendix D Abutment Scour (continued)

Reach	River Sta	Profile	E.G. Elev (ft)	E.G. Slope (ft/ft)	W.S.E Crit (ft)	W.S. (ft)
Min Ch El (ft)	Q Left (cfs)	Vel Left (ft/s)	Q Channel (cfs)	Vel Chnl (ft/s)	Q Right (cfs)	Vel Right (ft/s)
Top Width (ft)	Area Left (sq ft)	Area Channel (sq ft)	Area Right (sq ft)			
1	9.5	PF 1	14.11	0.002314	13.30	12.07
3.45	4863.20	3.52	22374.43	8.10	2762.37	3.82
2294.95	2341.72	2762.74	2341.72			
1	7.5	PF 1	13.64	0.002003	12.96	11.70
3.05	5329.25	3.23	21132.85	7.58	3537.90	3.47
2298.07	2400.92	2787.72	2400.92			

TYPE OF SCOUR

Evaluate the type of scour, Live-bed or clear water:

Clear-water Scour

Clear-water scour occurs when the approach flow is not transporting sediment. This occurs when the velocity of the flow under consideration is less than the tractive shear velocity. This will usually occur in overbank areas.

Live-Bed Scour

Live-bed scour occurs when the approach flow is transporting sediment. This occurs when the velocity of the flow under consideration is greater than the tractive shear velocity of the sediment.

Critical Velocity:

$V_c = 11.17\ (d^{0.167})\ D_{50}^{0.33}$ Bed material is sand with a d_{50} of 0.0066 ft.

For main channel flow, station 1300 to station 1400, from Table 2, the hydraulic depth is 8.06 ft

$V_c = 11.17\ (8.06)^{\wedge 0.167}\ (0.0066)^{\wedge 0.33}$

$V_c = 2.97$ ft/sec. For sections 1 through 4, the left overbank, the existing velocity is less than the critical velocity. For the main channel the velocity is greater than the critical velocity, therefore the abutment scour is clear water scour.

Appendix D Abutment Scour (continued)

By Froehlich's Method:

$y_s/ y_a = 2.27 * K_1 * K_2 * (L'/y_a)^{0.43} * Fr^{0.61} + 1$

Where: K_1 = coefficient for abutment shape
K_2 = coefficient for angle of embankment to flow
L' = length of active flow obstructed by the embankment
y_a = average depth of flow on the floodplain
Fr = Froude number of approach flow upstream of the abutment

For our conditions: From section 21 with bridge in place:

Pos Flow	Left Sta(ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P(ft)	Convey. %	Hydr Depth (ft)	Vel. (ft/s)
1 LOB	0	170.0	0	26.8	536.7	0	0.73	0
2 LOB	170.0	402.5	1260.2	618.8	232.5	4.20	2.66	2.04
3 LOB	402.5	635.0	1514.6	691.0	232.5	5.05	2.97	2.19
4 LOB	635.0	867.5	1883.0	787.4	232.5	6.28	3.39	2.39
			4657.8	2124.0				

Unit flow adjacent to left abutment = 1883.0/232.5 = 8.10 cfs/ft.

Pos Flow	Left Sta(ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P(ft)	Convey. %	Hydr Depth (ft)	Vel. (ft/s)
10 ROB	1500.0	1726.6	2541.4	930.2	226.7	8.47	4.10	2.73
11 ROB	1726.6	1953.3	1860.2	771.4	226.7	6.20	3.40	2.41
12 ROB	1953.3	2180.0	1501.6	678.4	226.7	5.01	2.99	2.21
13 ROB	2180.0	2390.0	0	579.9	210.0	0	2.76	0
14 ROB	2390.0	2600.0	0	115.5	76.7	0	1.51	0
			5903.2	3075.4				

Unit flow adjacent to right abutment = 2541.4/226.5 = 11.22 cfs/ft.

For Left abutment:
The obstructed flow, Q_{obl} = 4657.8 cfs. The obstructed area = 2124.0 sq.ft.
K_1 = 1, vertical face abutment
K_2 = 1, embankment perpendicular to flow
L' = length of active flow obstructed by the embankment, = 4657.8/8.10 = 575.0'
y_a = average depth of flow on the floodplain, A/L = 2124.0/575.0 = 3.69'
L'/ y_a = 575.0/3.69 = 156. **Greater than 25, use L'/ y_a = 25**
Fr = Froude number of approach flow upstream of the abutment:
V=Q/A = 4657.8/2124.0 = 2.19 ft/sec.
$Fr = V/(g * y_a)^{0.5}$ = 2.19/(32.2*3.69) $^{\wedge 0.5}$ = 0.201

Appendix D Abutment Scour (continued)

By Froehlich's Method: (continued)

$y_s / y_a = 2.27 * K_1 * K_2 * (L'/y_a)^{0.43} * Fr^{0.61} + 1 = 2.27*1*1*(25)^{0.43}*(0.201)^{0.61} + 1$
$y_s / y_a = 2.27*3.99*0.376 + 1 = 3.41+1 = 4.41$
$y_s = 4.41*y_a$
$= 4.41*3.69 = 16.3'$
At Left abutment: $y_s = 16.3'$

For Left abutment:
If L'/y_a is not limited to 25, but the actual is used, $L'/y_a = 156$,

then $y_s / y_a = 2.27*156^{\wedge 0.43}*0.376 + 1 = 7.49+1 = 8.49$

$= 8.49*3.67 = 31.2'$

For the right abutment:
The obstructed flow, Q_{obr} = 5903.2 cfs. The obstructed area = 3075.4 sq.ft.
K_1= 1, vertical face abutment
K_2= 1, embankment perpendicular to flow
L' = length of active flow obstructed by the embankment, = 5903.2/11.22 = 526.1'
y_a = average depth of flow on the floodplain, A/L = 3075.4/526.1 = 5.85'
L'/y_a = 526.1/5.85 = 90. **Greater than 25, use $L'/y_a = 25$**
Fr = Froude number of approach flow upstream of the abutment:

V=Q/A = 5903.2/3075.4 = 1.92 ft/sec.

$Fr = V/(g* y_a)^{0.5} = 1.92/(32.2*5.85)^{\wedge 0.5} = 0.140$
$y_s / y_a = 2.27 * K_1 * K_2 * (L'/y_a)^{0.43} * Fr^{0.61} + 1 = 2.27*1*1*(25)^{0.43}*(0.140)^{0.61} + 1$
$y_s / y_a = 2.27*3.99*0.301 + 1 = 2.73+1 = 3.73$
$y_s = 3.73*y_a$
$= 3.73*5.85 = 21.8'$
At right abutment: $y_s = 21.8'$

If L'/y_a is not limited to 25, but the actual is used, $L'/y_a = 90$,

then $y_s / y_a = 2.27*90^{\wedge 0.43}*0.301 + 1 = 4.73+1 = 5.73$

$= 5.73*5.85 = 33.5'$

Appendix D Abutment Scour (continued)

HIRE: For HIRE, scour is a function of the depth and Froude number for the flow adjacent to the abutment.

$y_s = 4.0 * K_1/0.55 * K_2 * Fr^{0.33}\ y_a$

At left abutment: y_a = 3.76, V = 4.09. Fr = 0.372

$y_s = 4.0*(1.0/0.55)*1.0\ (0.372)^{\wedge 0.33}\ (3.76) = 7.273*0.722*3.76 = 19.7'$

At right abutment: y_a = 4.35, V = 4.50. Fr = 0.380

$y_s = 4.0*(1.0/0.55)*1.0\ (0.380)^{\wedge 0.33}\ (4.35) = 7.273*0.726*4.35 = 23.0'$

"Pier scour"

For these analyses,
As the bank erodes, the abutment may be exposed to flow as a pier, scour is also computed for this condition.

Assume the abutment is supported on two shafts that act independently. Say 5' or 7' diameter shafts at 25 foot spacing. Use the hydraulic parameters for flow through the bridge (Station 1300 to Station 1400). Hydraulic depth = 8.91 ft. and velocity =9.34 ft/sec. Channel low point elevation 4.35.

Scour at a simple round pier.

a.) Since the depth of flow is less than 12 feet the scour is predicted for only the case with debris. The scour elevation is measured from bottom of debris, the stream bed.

Pier Geometry: Diameter=5.0 ft or 7.0 ft, single round shaft, K_1=1
Debris=4.0', K_1=1.1

Flow variables: Y_1=8.91 ft. V=9.34 ft./sec.

Angle of attack: ADOT minimum=15 degrees.

Effective width: a= 5.0'+4.0'=8.0 ft.

Froude Number: Fr $=v/((G*Y_1)^{0.5})$
$=(9.34)/((32.2*8.91)^{0.5})=0.55$

For 5' shaft: consider width with debris.
Effective width: a= 5.0'+4.0'=9.0 ft.

$Y_s/Y_1=2.0*K_1*K_2*((a/Y_1)^{0.65})*((Fr^{0.43})$

Appendix D Abutment Scour (continued)

"Pier scour" (continued)

For 5' shaft: consider width with debris.

$Y_S/8.9=2.0*(1.1)*(1.0)*((9.0/8.91)^{0.65})*(0.55^{0.43})$
$Y_S=8.9*(2.2)*(1.0)*(0.77)$
$Y_S=15.1$

For 7' shaft:
Effective width: a= 7.0'+4.0'=11.0 ft.

$Y_S/Y_1=2.0*K_1*K_2*((a/Y_1)^{0.65})*((Fr^{0.43})$
$Y_S/8.9=2.0*(1.1)*(1.0)*((11.0/8.91)^{0.65})*(0.55^{0.43})$
$Y_S=8.9*(2.2)*(1.147)*(0.77)$
$Y_S=17.3'$

Diameter	Scour	Elevation
5	15.1	-10.65
7	17.3	-12.85

Summary – Q = 30,000 cfs.

	Left Abutment		Right Abutment		Thalweg	
Ground Elev.	11.5		10.4		4.3	
SCOUR						
Method	Depth	Elev.[3]	Depth	Elev.[3]	Depth	Elev.[3]
Froehlich[1]	16.3	-12.0	21.8	-17.5		
HIRE:	19.7	-15.4	23.0	-18.7*		
5' "Pier Scour"					15.1	-10.8*
7' "Pier Scour"					17.3	-13.0*
Froehlich[2]	31.2	-26.9	33.5	-29.2		

* Value for design, depending on "pier diameter", use lower elevation from either HIRE or "Pier Scour".

NOTES: 1. Froehlich's with L'/y_a limited to 25.
2. Froehlich's with L'/y_a equal to actual value.
3. Elevations are calculated by subtracting scour depth from thalweg elevation.

Appendix D Abutment Scour (continued)

For Superflood of 51,000 cfs.
Unencroached condition:
Flow distribution:

From HEC-RAS for existing conditions at Section 14+50, the water surface elevation is 15.38, depth of flow = 15.38-4.45 = 10.93.

Table 5
HEC-RAS
Without bridge

Slice ID.	Left Station	Right Station	Flow	Area	Velocity
1	0	220	309.5	170.6	2.29
2	220	440	2222.4	679.1	3.27
3	440	660	2523.5	732.9	3.44
4	660	880	3056.7	822.5	3.72
5	880	1100	4060.0	974.9	4.16
6	1100	1200	4621.2	650.0	7.11
7	1200	1300	8412.5	931.2	9.03
8	1300	1400	8635.8	945.9	9.13
9	1400	1500	4828.8	667.4	7.23
10	1500	1720	4059.9	974.9	4.16
11	1720	1940	3053.0	821.7	3.72
12	1940	2160	2522.7	732.8	3.44
13	2160	2380	2222.4	679.1	3.27
14	2380	2600	390.5	170.6	2.29

Q_l=12,253 cfs; Q_{mc}= 24,498 cfs; Q_r =12,249 cfs
A_l= 3379.8 sq. ft. A_{mc}= 3194.5 sq. ft. Q_r = 3379.1 sq. ft.

Since the average velocity in the overbanks is greater than 2.97 ft/sec, the critical velocity, there is sediment transport. Look at “live-bed” scour.

Encroached Section:

At the left abutment: the obstructed flow, Station 0 to Station 850.
Q_{obl} = 12,253-4060-(30/220)*(3056.7) = 7776.2 cfs.
The obstructed area = 3379.8-974.92-(30/220)*(822.25) = 2292.7 sq.ft.

At the right abutment the obstructed flow, station 1500 to station 2600 is Q_{obr} = 12,249 cfs.
The obstructed area = 3379.1 sq.ft.

The distance from the bridge to the upstream section is 30 feet. Use a 1:1 contraction ratio, therefore at River Station 14.50, set the ineffective flow at 820 and 1530. At section 21.0 the encroachment stations are 170 and 2180. For the downstream section use an expansion coefficient of 2:1, this is from the expansion coefficient table for b/B=0.25, slope = 10 feet/mile and Nob/Nmc=1.3. Range of expansion coefficients is 1.3 to 2.0.

Appendix D Abutment Scour (continued)

Running HEC-RAS for the encroachments defined above results in the following conditions through the bridge opening:

Table 6
HEC-RAS
650' Bridge

Water Surface elevation 14.83,

Left Station	Right Station	Discharge	Hydraulic Depth	Velocity
820	1100	9642	5.98	5.76
1100	1200	7508	8.16	9.20
1200	1300	12,295	10.98	11.20
1300	1400	12,571	11.13	11.30
1400	1500	7776	8.34	9.33
1500	1530	1208	6.57	6.13

By Froehlich's Method:

$y_{s/}\ y_a = 2.27 * K_1 * K_2 * (L'/y_a)^{0.43} * Fr^{0.61} + 1$

Where: K_1= coefficient for abutment shape

K_2 = coefficient for angle of embankment to flow

L' = length of active flow obstructed by the embankment

y_a = average depth of flow on the floodplain

Fr = Froude number of approach flow upstream of the abutment

For our conditions: From section 21 with bridge in place:

Pos Flow	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P (ft)	Convey. %	Hydr Depth (ft)	Vel. (ft/s)
1 LOB	0	170.0	0	209.5	102.44	0	2.05	0
2 LOB	170.0	402.5	3296.8	1230.0	232.5	6.46	5.29	2.68
3 LOB	402.5	635.0	3625.7	1302.2	232.5	7.11	5.60	2.78
4 LOB	635.0	867.5	4084.0	1398.6	232.5	8.01	6.02	2.92
			11006.5	4140.3				

Unit flow adjacent to left abutment = 4084.0/232.5 =17.57 cfs/ft.

Pos Flow	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P (ft)	Convey. %	Hydr Depth (ft)	Vel. (ft/s)
10 ROB	1500.0	1726.6	4810.6	1526.0	226.7	9.43	6.73	3.15
11 ROB	1726.6	1953.3	4005.4	1367.2	226.7	7.85	6.03	2.93
12 ROB	1953.3	2180.0	3561.7	1270.2	226.7	6.98	5.62	2.80
13 ROB	2180.0	2390.0	0	1131.9	210.0	0	5.39	0
14 ROB	2390.0	2600.0	0	403.4	142.5	0	2.83	0
			12377.7	5698.7				

Unit flow adjacent to right abutment = 3561.7/226.5 = 15.72 cfs/ft.

Appendix D Abutment Scour (continued)

Froehlich's Method: (continued)

For Left abutment:
The obstructed flow, Q_{obl} = 11006.5 cfs. The obstructed area = 4140.3 sq.ft.

K_1= 1, vertical face abutment
K_2 = 1, embankment perpendicular to flow
L' = length of active flow obstructed by the embankment, = 11006.5/17.57 = 626.4'
y_a = average depth of flow on the floodplain, A/L = 4140.3/626.4 = 6.61'
L'/ y_a = 626.4/6.61 = 94.8. Greater than 25, use L'/ y_a = 25
Fr = Froude number of approach flow upstream of the abutment:
V=Q/A = 11006.5/4140.3= 2.66
Fr = $V/(g^* y_a)^{0.5}$ = 2.66/(32.2*6.61) ^0.5 = 0.182
$y_s/ y_a = 2.27*K_1*K_2 *(L'/y_a)^{0.43}*Fr^{0.61}+1 = 2.27*1*1*(25)^{0.43}*(0.182)^{0.61} + 1$
y_s/ y_a = 2.27*3.99*0.354 +1 = 3.21+1 = 4.21
$y_s = 4.21*y_a$
= 4.41*6.61 = 29.2'
At Left abutment: y_s = 29.2'

If L'/ y_a is not limited to 25, but the actual is used, L'/ y_a = 94.8
then y_s/ y_a = 2.27*94.8^0.43*0.354 +1 = 5.70+1 = 6.70
= 6.70*6.61 = 44.3'

For the right abutment:
The obstructed flow, Q_{obr} = 12377.7 cfs. The obstructed area = 5698.7 sq.ft.

K_1= 1, vertical face abutment
K_2= 1, embankment perpendicular to flow
L' = length of active flow obstructed by the embankment, = 12377.7/15.72 = 787.4'
y_a = average depth of flow on the floodplain, A/L = 5698.7/787.4 = 7.24'
L'/ y_a = 787.4/7.24 = 108.7. Greater than 25, use L'/ y_a = 25
Fr = Froude number of approach flow upstream of the abutment:
V=Q/A = 12377/5698.7 = 2.17
Fr = $V/(g^* y_a)^{0.5}$ = 2.17/(32.2*7.24)^0.5 = 0.142

$y_s/ y_a = 2.27*K_1*K_2 *(L'/y_a)^{0.43}*Fr^{0.61}+1 = 2.27*1*1*(25)^{0.43}*(0.142)^{0.61} + 1$
y_s/ y_a = 2.27*3.99*0.304 +1 =2.75+1 = 3.75
y_s =3.75*y_a
= 3.75*7.24 = 27.2'
At right abutment: y_s = 27.2'

Appendix D Abutment Scour (continued)

Froehlich's Method: (continued)

If L'/y_a is not limited to 25, but the actual is used, $L'/y_a = 109$,

then $y_s/y_a = 2.27*109^{\wedge 0.43}*0.304 + 1 = 5.19+1 = 6.19$

$= 6.19*7.24 = 44.8'$

HIRE: For HIRE, scour is a function of the depth and Froude number for the flow adjacent to the abutment.

$y_s = 4.0*K_1/0.55*K_2 *Fr^{0.33}\ y_a$

At left abutment: $y_a = 5.98$, V = 5.76. Fr = 0.415

$y_s = 4.0*(1.0/0.55)*1.0*(0.415)^{\wedge 0.33}\ (5.98) = 7.273*0.748*5.98 = 32.5'$

At right abutment: $y_a = 6.57$, V = 6.13. Fr = 0.421

$y_s = 4.0*(1.0/0.55)*1.0\ (0.421)^{\wedge 0.33}\ (6.57) = 7.273*0.752*6.57 = 35.9'$

"Pier scour"

For these analyses,
As the bank erodes, the abutment may be exposed to flow as a pier, scour is also computed for this condition.

Assume the abutment is supported on two shafts that act independently. Say 5' or 7' diameter shafts at 25 foot spacing. Use the hydraulic parameters for flow through the bridge. Hydraulic depth = 11.13 ft. and velocity =11.3 ft/sec. Channel low point elevation 4.35.

Scour at a simple round pier.

b.) Since the depth of flow is less than 12 feet the scour is predicted for only the case with debris. The scour elevation is measured from bottom of debris, the stream bed.

Pier Geometry: Diameter=5.0 ft or 7.0 ft, single round shaft, K_1=1
Debris=4.0', K_1=1.1

Flow variables: Y_1=11.13 ft. V=11.3 ft./sec.

Angle of attack: ADOT minimum=15 degrees.

Froude Number: Fr $=v/((G*Y_1)^{0.5})$

$=(11.3)/((32.2*11.13)^{0.5})=0.60$

Appendix D Abutment Scour (continued)

“Pier scour” (continued)

$Y_S/Y_1=2.0*K_1*K_2*((a/Y_1)^{0.65})*((Fr^{0.43})$

For 5’ shaft: consider width with debris.
Effective width: a= 5.0’+4.0’=9.0 ft.

$Y_S/11.13=2.0*(1.1)*(1.0)*((9.0/11.13)^{0.65})*(0.60^{0.43})$
$Y_S=11.13*(2.2)*(1.0)*(0.87)*(0.80)$
$Y_S=17.0$

For 7’ shaft:
Effective width: a= 7.0’+4.0’=11.0 ft.

$Y_S/Y_1=2.0*K_1*K_2*((a/Y_1)^{0.65})*((Fr^{0.43})$
$Y_S/11.13=2.0*(1.1)*(1.0)*((11.0/11.13)^{0.65})*(0.60^{0.43})$
$Y_S=11.13*(2.2)*(0.99)*(0.80)$
$Y_S=19.4’$

Summary – Q = 51,000 cfs

	Left Abutment		Right Abutment		Thalweg	
Ground Elev.	11.5		10.4		4.3	
SCOUR						
Method	Depth	Elev.[3]	Depth	Elev.[3]	Depth	Elev.[3]
Froehlich[1]	29.2’	-24.9	27.2’	-22.9		
HIRE:	32.5’	-28.2*	35.9’	-31.6*		
5’ “Pier Scour”					17.0	-12.7
7’ “Pier Scour”					19.4	-15.1
Froehlich[2]	44.3’	-40.0	44.8’	-40.5		

*** Value for design, use lower elevation from HIRE**

NOTES: 1. Froehlich’s with $L’/y_a$ limited to 25.
2. Froehlich’s with $L’/y_a$ equal to actual value.
3. Elevations are calculated by subtracting scour depth from thalweg elevation.

Appendix E Pier Scour

- Simple Round Pier w/ debris
- Stem wall Pier w/ debris
- Simple Round Pier w/ flow depth >debris

Example 1. Scour at a simple round pier w/ debris.

Pier Geometry: Diameter=8.0 ft, single round shaft.
Debris=4.0 ft.
K_1=1.1, K_2 = 1.0, K_3 = 1.1, K_4 = 1.0

Flow variables: Y_1=10.2 ft. V=11.02 ft./sec.
Debris is for full depth, Y_1<12 ft.
Streambed elevation, 100.0

Angle of attack: ADOT minimum=15 degrees.

Effective width: a= 8.0’+4.0’=12.0 ft.

Froude Number: Fr $=v/((G*Y_1)^{0.5})$
$=(11.02)/((32.2*10.2)^{0.5})=0.63$

$Y_S/Y_1=2.0*K_1*K_2*K_3*K_4*((a/Y_1)^{0.65})*((Fr^{\wedge 0.43})$
$Y_S/10.2=2.0*(1.1)*(1.0)*(1.1)*(1.0)*((12.0/10.2)^{0.65})*(0.61^{0.43})$
$Y_S=10.2*(2.42)*(1.13)*(0.81)$
$Y_S=22.6$
Scour elev. = 100.0-22.6=77.4

Example 2. Scour at a stem wall pier.

Pier Geometry: Stem wall thickness=1.0 ft, length=48 ft.
Debris=4.0 ft., a= 1’+4’ = 5’
K_1=1.1, K_3 = 1.1, K_4 = 1.0
L/a=48/5=9.6

Angle of attack: ADOT minimum=15 degrees. Due to site conditions, use 20 degrees
$K_2=(\cos\Omega+(L/a)*\sin\Omega)^{0.65}$
$K_2=(\cos(20)+(9.6)*\sin(20))^{0.65}$
$K_2=(0.940+9.6*0.342)^{0.65} = (4.22)^{0.65}$
$K_2=2.55$

Flow variables: Y_1=10.2 ft. V=11.02 ft./sec.
Debris is for full depth, Y_1<12 ft.
Stream bed at elev, 100.0

Appendix E Pier Scour (Continued)

Example 2. Scour at a stem wall pier. (continued)

Froude Number: Fr $=v/((G*Y_1)^{0.5})$
$=(11.02)/((32.2*10.2)^{0.5})=0.63$

$Y_S/Y_1=2.0*K_1*K_2*K_3*K_4*((a/Y_1)^{0.65})*((Fr^{\wedge 0.43})$

$Y_S/10.2=2.0*(1.1)*(2.55)*(1.1)*(1.0)*((5/10.2)^{0.65})*(0.61^{0.43})$
$Y_S=10.2*(6.17)*(0.63)*(0.81)$
$Y_S=10.2*3.15 = 32.1$ ft.
Scour elev. = 100.0-32.1= 67.9 ft

Example 3. Scour at a simple round pier. Depth of flow greater than debris depth.

Scour is predicted for two cases: a.) based on debris width measured from bottom of debris and b.) without debris from stream bed.

Pier Geometry:Diameter=4.0 ft, single round shaft without debris, $K_1=1$
Debris=4.0’, single round shaft with debris, $K_1=1.1$
$K_3 = 1.1$, $K_4 = 1.0$

Flow variables: $Y_1=15.0$ ft. V=11.02 ft./sec.
Debris is less than full depth, $Y_1>12$ ft.

Stream bed at elev, 100.0

Angle of attack: ADOT minimum=15 degrees.

Effective width: a= 4.0’+4.0’=8.0 ft.

Froude Number: Fr $=v/((G*Y_1)^{0.5})$
$=(11.02)/((32.2*10.2)^{0.5})=0.63$

Calculate two scour depths.

A.) consider width with debris.
Measure scour depth from bottom of debris.
$Y_S/Y_1=2.0*K_1*K_2*K_3*K_4*((a/Y_1)^{0.65})*((Fr^{0.43})$
$Y_S/15.0=2.0*(1.1)*(1.0)*(1.1)*(1.0)*((8.0/15.0)^{0.65})*(0.61^{0.43})$
$Y_S=15.0*(2.42)*(0.66)*(0.81)$
$Y_S=19.4$
Scour elev. = 100.0+(15-12)-19.4= 83.6

Appendix F SCOUR DESIGN INFORMATION

The ADOT Bridge Group has prepared the **LRFD Bridge Design Guidelines**. Included in the manual is a requirement for structural analysis of the effects of scour. To properly implement this requirement, estimates of scour must be provided to bridge design personnel by the drainage engineer.

Two Stages of Flow shall have scour calculated. They are:

1. Design Frequency Level (Case 3)
   This shall include the long term degradation, and the general and local scour for flows up to and including the design event. Bank protection shall be designed to withstand this scour occurrence. Debris effects shall be included in the scour calculations.

2. Maximum Expected Flood Level (SUPERFLOOD) (Case 4)

   This shall include the long term degradation, and the general and local scour for flows up to and including the maximum expected flood event. The goal of this level of design is that the bridge structure is still standing and usable, with repair or replacement of approach embankments, if necessary. Bank protection may have failed. Debris effects shall be included in the scour calculations.

The drainage engineer shall anticipate the data needs of the bridge designer and provide the necessary information in a clear and concise format. This should include the following items:

1. The existing channel thalweg (elevation) (Case 1).
2. Water surface elevation for Design frequency flood (Case 3)
3. Long term degradation (including headcut movement from existing gravel mining or allowance for future gravel mining) Case 3).
4. General and local scour at the design flow event (Case 3).
5. Water surface elevation for the maximum expected flood (Case 4)
6. General and local scour at the maximum expected flood level (Case 4).